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AMERICAN CONCRETE INSTITUTE

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Technical pages of the ACI JOURNAL offer a medium for reporting what is new and interesting in concrete research, design, construction, maintenance, or manufacturing. ACI members and JOURNAL readers are urged to share their knowledge and experience by submitting papers and reports for publication in the ACI JOURNAL. Time and changing interests demand that new contributors with new ideas take their place beside the old.

●

The Technical Activities Committee selects the papers, committee reports, discussions, and other contributions for JOURNAL publication. A volunteer corps of experts in various segments of the field assists in technical appraisal of the manuscripts. Technically speaking, acceptable contributions present original material for improvement of design, manufacturing, or construction; confirm, revise, or upset established ideas or practices; or review, digest, or arrange accumulated experience for more ready use.

Further questions raised by TAC: will the contribution fit into a balanced publication schedule? Can it be published at reasonable expense to the Institute?

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Contributors should furnish glossy prints or inked tracings of illustrations along with manuscripts submitted in triplicate for publication. Details of the form and physical arrangement of contributions are given in the 1955 *ACI Directory*, p. 14. Separate copies of this "publications policy" are available free on request to the Secretary.

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In ACI Journal next month

SUBCOMMITTEE II of ACI COMMITTEE 622 presents "Form Construction Practices," a report covering construction practices and recommendations for the improvement of general industry standards to promote safety, economy, and good workmanship. A summary of answers to questionnaires sent to contractors engaged in all types of concrete construction is presented in an appendix.

★

"A Study of Plastic Hinging at the Intersection of Beams and Columns" by GEORGE C. ERNST reveals the amount of concentrated plastic rotation developed at the connection between beams and columns. Thirty-three tests were conducted at various loading rates, and with varying steel ratios, column widths, and nominal concrete strengths.

★

ROY R. CLARK describes "Mass Concrete Control in the Detroit Dam" where control procedures established uniformly stable conditions at the base of the dam. The project was constructed substantially without cracks on ends or surfaces of blocks, at a total cooling cost of $0.63 per cu yd of mass concrete. Economy was achieved by carefully coordinated planning during the design period and by adopting several complementary construction techniques.

★

Inaugurating the new ACI publication policy, discussion of papers published in the July through December 1956 JOURNALS will appear as the last section in the June 1957 issue. Discussions of the following are scheduled:

"Prestressed Concrete Pavements for Airfields" by THOMAS CHOLNOKY

"Spacing of Reinforcement in Beams" by S. J. CHAMBERLIN

"Some Implications of Recent Diagonal Tension Tests" by PHIL M. FERGUSON

"An Expression for Creep and Its Application to Prestressed Concrete" by CEVDET Z. ERZEN

"Empirical Time-Strength Relations of Concrete" by MYRON L. GORAL

"Strength of a Concrete Slab Prestressed in Two Directions" by A. C. SCORDELIS, K. S. PISTER, and T. Y. LIN

"Auditorium Framed with Prestressed Roof Girders" by ERIC C. MOLKE

"Structural Lightweight-Aggregate Concrete" by RALPH W. KLUGE

"Helicoidal Staircases of Reinforced Concrete" by VICTOR R. BERGMAN

"Considerations in the Selection of Slab Dimensions" reported by SUBCOMMITTEE II, ACI COMMITTEE 325

"Guide for Ultimate Strength Design of Reinforced Concrete" by CHARLES S. WHITNEY and EDWARD COHEN

"Lightweight Aggregates for Concrete Masonry Units" by C. C. CARLSON

"Insulating Concretes" by R. C. VALORE, JR.

"Prestressed Bridge Designed for Crane Load at Niagara River Weir" by A. M. LOUNT

"Particle Interference in Concrete Mixes" by B. J. BUTCHER and H. J. HOPKINS

"Proposed Recommended Practice for Evaluation of Compression Test Results of Field Concrete" reported by ACI COMMITTEE 214

Title No. 53-57

Hot Weather Concreting Problems*

Progress report of ACI Committee 605

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SYNOPSIS

This progress report discusses the subject matter which is proposed for inclusion in a recommended practice now being drafted by the committee. It discusses, briefly, effects of concreting in hot weather; factors in minimizing hot weather difficulties; keeping concrete temperature down by controlling the temperature of its ingredients; production and delivery, placement and protection, and concrete testing in hot weather.

INTRODUCTION

That hot weather creates special problems of concreting there can be no doubt. This seems to have come as a surprise to some. That there should be cold weather difficulties—yes! But hot weather, why?

Actually, the over-all problems have been recognized for a long time, but not in the experience of the average producer and user of concrete. Emphasis has been placed on hot concrete rather than hot weather—and primarily from the point of view of heat of hydration in massive sections. There has been occasional recognition of hot weather as such, with placement of concrete restricted to the cool of the evening and night. These have been looked upon as special cases and not of general application.

*Presented at the ACI 53rd annual convention, Dallas, Tex., Feb. 26, 1957. Title No. 53-57 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 28, No. 11, May 1957, *Proceedings* V. 53. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1957. Address P.O. Box 4754, Redford Station, Detroit 19, Mich.

This report was submitted to letter ballot of the committee which consists of 17 members; 13 members returned their ballots, of whom 13 have voted affirmatively.

The problem is made more pressing in recent years by ever-increasing speed of construction. We have higher early-strength cements which means finer grinding and modifications in compound composition contributing to rapid gain in strength and rapid generation of heat. Larger batches of concrete are handled which, undoubtedly, tend to accentuate the problem.

ORGANIZATION OF COMMITTEE

Late in 1955, Committee 605 was organized "to review information on the effect of hot weather on the characteristics of fresh and hardened concrete and to develop a recommended practice for controlling ingredients, batching, mixing, transporting, placing, protecting, and curing concrete during hot weather." The committee held its first meeting in Philadelphia on Feb. 23, 1956. Its assignment was reviewed and a great deal of correspondence ensued.

Four subcommittees were appointed dealing with various aspects of the problem, as follows: (1) Materials, C. E. Wuerpel, chairman; (2) Production and Delivery, Byron P. Weintz, chairman; (3) Properties and Tests, Ernst Gruenwald, chairman; and (4) Placement and Protection, Lewis H. Tuthill, chairman. There is, of course, a great deal of overlapping in the scope of the subcommittees' work and, in this report, no effort is made to distinguish among their contributions.

EFFECTS OF CONCRETING IN HOT WEATHER

Among the principal evidences that mixing and placing concrete in hot weather present problems are:

1. Strength is frequently reduced. To a large degree, this may be due to increased water demand which accompanies high temperature and which results for the most part from the more rapid hydration of the cement and perhaps in a small degree the reduced surface tension of the water. Another factor contributing to lower strength is, undoubtedly, the greater importance of adequate protection and curing under conditions conducive to rapid evaporation. Further, there is evidence that test cylinders, even when adequately protected from drying out, suffer strength reductions due to exposure to relatively high temperature during early periods.[1,10,11]

2. The formation of cracks in flat surfaces at early ages is much more prevalent in hot weather than at other times. Plastic shrinkage cracks which form in the partially hardened concrete within a few hours after placement develop most frequently, although not exclusively, under hot weather conditions. Rapid drying of flat surfaces is a principal cause of this plastic shrinkage, and hot weather is a heavy contributor to such conditions.[13]

3. Concrete that sets at high temperature has greater thermal shrinkage than that setting at low temperature. This, coupled with the effects of drying, aggravates the problem of shrinkage cracking.

4. High temperatures accelerate the early hydration of the cement with a resulting increase in rate of stiffening of concrete. That, as already stated, increases the demand for mixing water which not only reduces strength but contributes to volume changes, including plastic shrinkage.

5. There are many miscellaneous problems, most of them related to the principal ones named above, which result from hot weather. The maintenance of adequate curing conditions is made more difficult. The amount of purposely entrained air may be less readily controlled due, primarily, to the reduced surface tension of the water. Certainly, for a given amount of air-entraining agent, warm concrete will entrain less air than cool concrete.[12,14]

FACTORS IN MINIMIZING HOT WEATHER DIFFICULTIES

Although a great deal remains to be learned, experience and research have provided some indications of methods for reducing the dangers of concreting in hot weather. Excessive water demand with its adverse effect on strength and volume changes, and the effects of high temperature per se on these characteristics, can be reduced by maintaining concrete temperatures at moderately low level. Problems of excess water demand and strength reduction may also be alleviated in some cases by the judicious use of admixtures.

Attention to scheduling and delivery to avoid excessive mixing or delays of discharge help to prevent drying and stiffening before placement, or the need for adding retempering water. Proper protection and curing after placement reduce the likelihood of cracking due to plastic shrinkage, drying, or cooling from high temperatures.

MATERIALS

The most direct approach to keeping concrete temperature down is by controlling the temperature of its ingredients. The contribution of each material in a concrete mixture to the temperature of the concrete is directly related to the temperature and specific heats of the materials and the quantities used. A good approximation of the temperature of fresh concrete can be calculated from established relationships.* In warm weather, the computed temperature is generally slightly lower than the actual temperature due to the fact that the computations do not take into account the heat of wetting, the heat liberated by hydration of the cement, and the mechanical work of mixing. Calculations based on the formula in the footnote show that, for a typical concrete containing 6 sacks of cement per cu yd with a water-cement

* $$T = \frac{S\,(T_a W_a + T_c W_c) + T_f W_f + T_m W_m}{S\,(W_a + W_c) + W_f + W_m}$$

where:

W_a = weight in lb of aggregate (surface dry) and T_a = its temperature in deg F
W_c = weight in lb of cement and T_c = its temperature in deg F
W_f = weight in lb of free moisture in aggregate and T_f = its temperature in deg F
W_m = weight in lb of mixing water and T_m = its temperature in deg F
S = 0.20 assumed specific heat of dry materials
T = temperature of freshly-mixed concrete, deg F

ratio of 0.5 by weight, the temperature would be changed approximately 1 F by any one of the following changes in temperature of ingredients: cement 9 F; water 3.6 F; aggregate 1.6 F.

From the preceding it will be seen that the aggregates and mixing water exert the most pronounced effect on the temperature of the concrete. Thus, it is essential to utilize all available means for maintaining these materials at low temperature for hot weather concreting.

Aggregates

There are several procedures that lower the temperature of the aggregates. Shading the stockpile from direct rays of the sun provides some benefit. Sprinkling stockpiles and keeping them moist results in cooling due to evaporation, this procedure being especially effective when the relative humidity is low; it is less effective with dry aggregates which absorb water. When aggregates are stockpiled during hot weather it is desirable for successive layers to be sprinkled as the pile is built up. On some large jobs[6] aggregates have been cooled by forcing refrigerated air through the bins.

Water

Pound for pound, the mixing water has the greatest effect of any of the ingredients on the temperature of the concrete since it has a specific heat about 4½ to 5 times those of cement and aggregates. It should not be overlooked, however, that it is used in much lesser quantities than other ingredients. Even so, the use of cool mixing water is effective and every effort should be made to use water from a cool source and to keep it cool by the protection of pipes and tanks.

The use of ice as a part of the mixing water greatly increases its effectiveness, since, on melting, it takes up heat at the rate of 144 btu per lb. Thus, if ice were used as 50 percent of the mixing water in the 6-sack concrete considered above, the melting of the ice alone would lower the concrete temperature approximately 20 F and the resulting water would have an additional cooling effect in the neighborhood of 7 to 8 F. To take advantage of the heat of fusion, the ice must be incorporated directly into the concrete as a part of the mixing water. Conditions must be such that the ice is completely melted by the time the mixing is completed.

Cement temperature

Hot cement may result from the heat generated mechanically during the grinding of the clinker and also from the grinding of hot clinker. Since this heat is lost slowly during the storage, cement is often delivered at relatively high temperature. Although, as already shown, changes in temperature of the cement produce significantly less change in the temperature of the fresh concrete than the other ingredients, it does exert an effect, and therefore, it is prudent to place a maximum limit on its temperature as it enters the concrete. A reasonable limit may be 170 F.

Admixtures

The hydration of cement is accelerated by increasing the temperature of the concrete. It seems probable that this acceleration is largely responsible for the increase in water requirement caused by increased temperature of the fresh concrete. This suggests the possibility that certain retarders, when used as admixtures, might offset the increase in water requirement at higher temperatures. If that is so, they should prove to be beneficial provided they do not interfere with the subsequent hydration of the cement and strength development of the concrete.

In tests reported by Tuthill and Cordon,[8] a group of retarders were found to delay setting time of the concrete and, in some cases, to increase the strength. The entire field of the use of admixtures that may prove to be beneficial for hot weather concreting practices deserves further study. Unless experience has been had with a retarder, its use as an admixture should not be attempted without technical advice, or, preferably, advance experiment with the cement and other concreting materials involved, to determine the extent of its effects on the setting time and other properties of the concrete.[9]

PRODUCTION AND DELIVERY

Problems of production and delivery overlap those of materials. The temperatures of the aggregates, water, and cement have an important bearing and every effort should be made to maintain them at such a level that they will not contribute unduly to high temperature of the concrete. The ready-mixed concrete operation presents some special problems. During hot weather, care should be exercised to reduce the amount of mixing done on the concrete to the minimum which will insure adequate quality. Effort should be made also to reduce the time element to the extent practical. Trucks should be dispatched so as to avoid delay and unnecessary additional mixing at the job. Exposure to the hot sun while waiting to be unloaded should be minimized. Heat reflecting colors for mixer drums should be expected to help.

A temperature of 90 F is considered a reasonable maximum limit for concrete at the time of placement in hot weather. However, it is cautioned that in many situations difficulty may be encountered with concrete at temperatures approaching 90 F and every effort should be made to keep the concrete at lower temperature.

PLACEMENT AND PROTECTION

Difficulties from hot weather concreting can be minimized by more rigid controls of placement and protection than are required at other times. In addition to controlling the temperature of the concrete as delivered to the work, much can be done with placement facilities and practices to keep temperatures low. To the extent feasible, mixers, chutes, belts, pump lines, and forms should be shaded. Surfaces of such equipment as pump lines, if painted white, will absorb substantially less heat than when painted other

colors. Aluminum painted surfaces, for example, may be 10 to 15 deg warmer in the hot sun than white surfaces. Surfaces covered with damp burlap, kept damp, offer many advantages; the heat absorption is low and cooling results from evaporation.

Sprinkling of forms, reinforcing, and subgrade with cool water just prior to placement is beneficial. Wetting down the area around the work will cool the surrounding air and increase its humidity, thus lessening the likelihood of rapid setting. When temperature conditions are critical, it may be prudent to restrict work to the cool of the evening and the night. Not only are temperatures lower but evaporation is less.

Placement

Speed of placement and finishing helps to minimize hot weather difficulties. Delays contribute to slump loss and the resultant use of additional mixing water to offset it. Ample personnel should be available to handle and place the concrete immediately after its delivery to the forms at an acceptable temperature. On flat work, all steps in finishing should be carried out promptly and here again adequate personnel, both as to number and skill, are important.

Other modifications in usual placement practices are helpful. Concrete should be placed in layers thin enough and in areas small enough so that vibration or other working of the concrete will insure complete union of adjacent layers. If cold joints tend to form, if surfaces set and dry too rapidly, or if plastic shrinkage cracks appear, the areas should be kept moist by means of fog spray, wet burlap, cotton mats, or other means. When these precautions are not sufficient, it may be desirable, as mentioned earlier, to supplement them by restricting work to the cool of the evening and the night. If plastic shrinkage cracks do form, they can often be worked together during the finishing operation.

In short, all placement procedures should be directed to keeping the concrete as cool as practical and assuring its setting and hardening under temperature conditions which are reasonably uniform and moisture conditions which will minimize drying and drying shrinkage. Concrete, however it is delivered, by pump, truck mixer, or otherwise, should be received at the forms at acceptable temperatures and whatever is practicable should be done to minimize increases in them.

In addition to the precautions outlined, further protection may be secured by the use of the proper amount of an approved water-reducing retarder. As already pointed out, the use of such admixtures should not be attempted without informed advice and advance experiment with the materials involved.

Protection and curing

Since hot weather leads to more rapid setting and drying of concrete, protection and curing are far more critical than during cool weather. Consequently, particular attention should be paid to having all facilities ready for prompt application of curing. Care should be exercised to protect exposed surfaces from drying for any reason. Wherever it is practicable, continuous

water curing is to be preferred. Continuity is important because volume changes due to alternations of wetting and drying are detrimental and promote the development of pattern cracking.

Reliance should not be placed on forms for curing during hot weather. Whenever practical, water should be applied to formed surfaces, while the forms are still in place. Unformed surfaces should be kept moist by wet curing for at least 24 hr—and the covering should be placed as soon as the concrete has hardened sufficiently to prevent surface damage. If moist curing is not continued beyond 24 hr, the surface should be sprayed with white pigmented sealing compound or covered with a suitable plastic membrane. The pigment not only reflects heat but simplifies checking the uniformity and extent of application of the membrane. The need for adequate curing is greatest during the first few days after placement.

On hardened concrete and on flat surfaces in particular, it is desirable that curing water initially applied should not be much cooler than the concrete because of the shrinkage stresses which would be introduced, with resultant cracking. Where backfilling is involved, it should be done as soon as safe and practicable. At the completion of controlled moist curing, an effort should be made to reduce the rate of drying by avoiding air circulation to the extent that is feasible.

CONCRETE TESTING

The accuracy of testing is always highly important. In hot weather, conformance to the standard test methods becomes more critical.

As discussed earlier, high temperature of concrete at early ages is detrimental to strength independently of the side effects of increased water requirements and the possibilities for rapid stiffening. In addition to the effects of heat, further damage to test specimens may result from loss of moisture due to drying. For these reasons, extra care is needed in hot weather to maintain temperature and moisture conditions for strength test specimens as required in the standard test methods.

Much that has been said concerning protection of concrete in the structure can be extended to apply to strength test specimens. Proper temperatures can be maintained by avoiding exposure to the sun and by utilizing the cooling effect of evaporating water, either from damp burlap or wet sand covering the specimens. Merely covering the test specimen with a lid or plate is not sufficient in hot weather to prevent loss of moisture. Damp sand, wet burlap or, in the rare cases when available, fog sprays will insure retention of water by the specimens.

Specimens used as the basis for acceptance must be transferred at the age of 1 day to a location (usually the laboratory) where they will receive continuous standard moist curing until test. This requirement, as in the case of provisions for early protection, is of particular importance in hot weather because of the great damage to strength which results from high temperatures and drying.

CLOSURE

It is hoped that the presentation of this progress report will inspire discussion which will be of assistance to the committee in outlining a recommended practice for hot weather concrete which will serve to promote higher quality of concrete placed during hot weather.

REFERENCES

1. *Concrete Manual*, Bureau of Reclamation, U. S. Department of Interior, 6th Edition, 1955, p. 464.

2. House, R. F., "The Inundation Method of Cooling Concrete Aggregates at Bull Shoals Dam," Office of the Chief of Engineers, U. S. Army Corps of Engineers, 1949; Concrete Conference, Mar. 13, 1949.

3. Roberts, H. H., "Cooling Materials for Mass Concrete," ACI JOURNAL, June 1951, *Proc.* V. 47, pp. 821-832.

4. Budge, J. J., "Tube Ice Chills Concrete Mix in New Australian Dam Construction," *Industrial Refrigeration*, V. 126, No. 4, Apr. 1954, pp. 15-17, Abstract in *Engineering Index*, 1954.

5. Ogden, M., "How to Keep your Dam Concrete Cool," *Diesel Progress*, V. 20, No. 6, June 1954, pp. 25-27, Abstract in *Engineering Index*, 1954.

6. Thomson, P., "Precooled Concrete in 'Chief Joe' Dam," *Refrigerating Engineer*, V. 62, No. 10, Oct. 1954, pp. 50, 104-105, Abstract in *Engineering Index*, No. 55-1898.

7. Irwin, Harry, "The Ice Man Cometh—The Problem of Cooling Concrete for Certain Uses During Extremely Hot Weather," *Concrete*, V. 64, No. 6, June 1956, pp. 29, 31. See also "Producing Cooled Concrete," by Harry Irwin, presented at the 26th Annual Convention of the National Ready Mixed Concrete Association.

8. Tuthill, Lewis H., and Cordon, William A., "Properties and Uses of Initially Retarded Concrete," ACI JOURNAL, Nov. 1955, *Proc.* V. 52, pp. 273-286.

9. "Admixtures for Concrete," ACI Committee 212, ACI JOURNAL, Oct. 1954, *Proc.* V. 51, pp. 113-148.

10. Lerch, William, "Hot Cement and Hot Weather Concrete Tests," Portland Cement Association, Chicago, 1955.

11. Bloem, D. L., "Effect of Curing Condition on Compressive Strength of Concrete Test Specimens," *Publication* No. 53, National Ready Mixed Concrete Association, Washington, D. C., 1954.

12. Walker, Stanton, and Bloem, D. L., "Design and Control of Air-Entraining Concrete," *Publication* No. 60, National Ready Mixed Concrete Association, Washington, D. C., 1955.

13. "Prevention of Plastic Cracking in Concrete," Portland Cement Association, Chicago, 1955.

14. Walker, Stanton, and Bloem, Delmar L., "Studies of Concrete Containing Entrained Air," ACI JOURNAL, June 1946, *Proc.* V. 42, pp. 629-640.

15. Lerch, William, "Plastic Shrinkage," ACI JOURNAL, Feb. 1957, *Proc.* V. 53, pp. 797-802.

Discussion of this report should reach ACI headquarters in triplicate by Sept. 1, 1957, for publication in the Part 2, December 1957 JOURNAL.

Title No. 53-58

Tables for Circularly Curved Horizontal Beams with Symmetrical Uniform Loads*

By MARTIN SCHULZ† and MAURICIO CHEDRAUI‡

SYNOPSIS

These tables will simplify and expedite the engineer's task of calculating circular beams without developing the formulas. Step-by-step derivation of the tables is given.

The circularly curved horizontal beam with fixed ends is statically indeterminate to the third degree. That is, at any section in such a beam there exist three unknowns which must be evaluated for the solution of the force system. A convenient set of such unknowns consists of flexural and torsional moments, Mf and Mt, and a shear force, Q. For the general case it is practical to analyze antisymmetrical load conditions.

However, circularly curved horizontal beams with symmetrical uniform loads only will be treated here. At the midsection B of such a beam (see Fig. 1), we have, by symmetry, only the statically indeterminate flexural moment X of unknown magnitude. If we divide moment X into two moments Xt_s (torsional) and Xf_s (flexural), acting in planes respectively parallel to and perpendicular to any section S at an angle ϕ from the midpoint, we may write for these moments:

$$Xt_s = \sin \phi \, X \quad \ldots\ldots (1)$$

$$Xf_s = \cos \phi \, X \quad \ldots\ldots (2)$$

The total flexural moment Mf_s acting at any sections S will then be the flexural component of the midpoint moment X less the flexural moment Mf_{qs} due to the uniform load q distributed from the midpoint B to section S:

$$Mf_s = \cos \phi \, X - Mf_{qs} \quad \ldots\ldots (3)$$

The total distributed load from midpoint B to section S is:

$$Q = q \, r \, \phi \quad \ldots\ldots (4)$$

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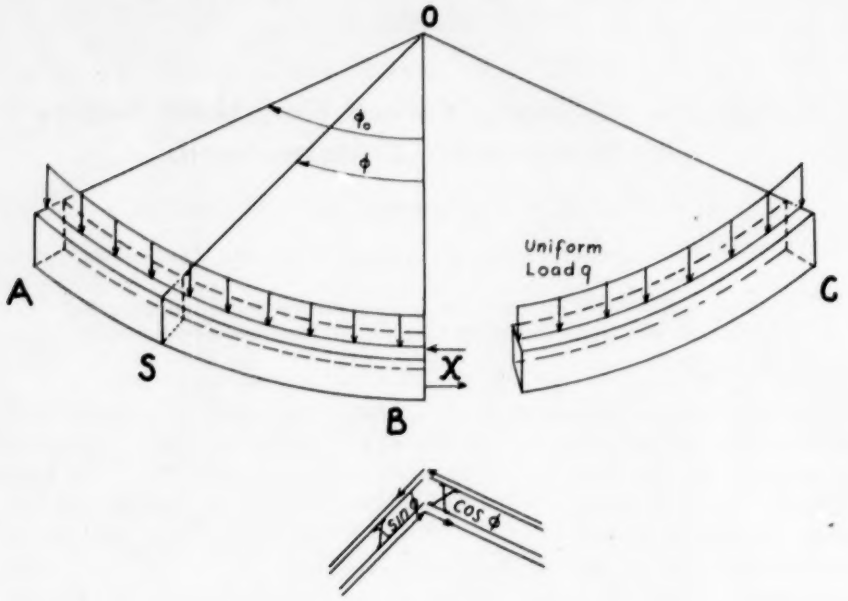


Fig. 1

The flexural moment produced by this load may be computed by considering it to be concentrated at the centroid of the arc *BS*. Referring to Fig. 2 for notation:

$$s = 2\,r \sin \phi/2 \quad (5)$$

$$b = r\,\phi \quad (6)$$

$$y = r\,s/b = 2\,r/\phi \sin \phi/2 \quad (7)$$

$$n = y \sin \phi/2 = 2\,r/\phi \sin^2 \phi/2 = r/\phi\,(1 - \cos \phi) \quad (8)$$

$$Mf_{qo} = Q\,n = q\,r^2\,(1 - \cos \phi) \quad (9)$$

From Eq. (3) and (9),

$$Mf_s = \cos \phi\,X - q\,r^2\,(1 - \cos \phi) \quad (10)$$

Similarly, Mt_s, the total torsional moment acting at section *S*, may be computed to be:

$$Mt_s = \sin \phi\,X - Mt_{qo} = \sin \phi\,X - q\,r^2\,(\phi - \sin \phi) \quad (11)$$

Castigliano's theorem may be stated as requiring that the distribution of forces in a loaded system is such that the work done in the system by the

applied loads is a minimum. The work done by the flexural moment from the midspan to the end of the beam is:

$$Uf = \int_0^{\phi_0} \frac{(Mf_s)^2}{EI} r\, d\phi \quad \text{.....(12)}$$

The work done by the torsional moments in the same interval is:

$$Ut = \int_0^{\phi_0} \frac{(Mt_s)^2}{GJ'} r\, d\phi \quad \text{.....(13)}$$

where J' is the equivalent torsional second moment of area of a rectangular section computed by St. Venant's formula:

$$J' = \frac{A^4}{40\,J} \quad \text{.....(14)}$$

$$J = \frac{ab^3 + ba^3}{12} \quad \text{(Polar second moment of area)} \quad \text{.....(15)}$$

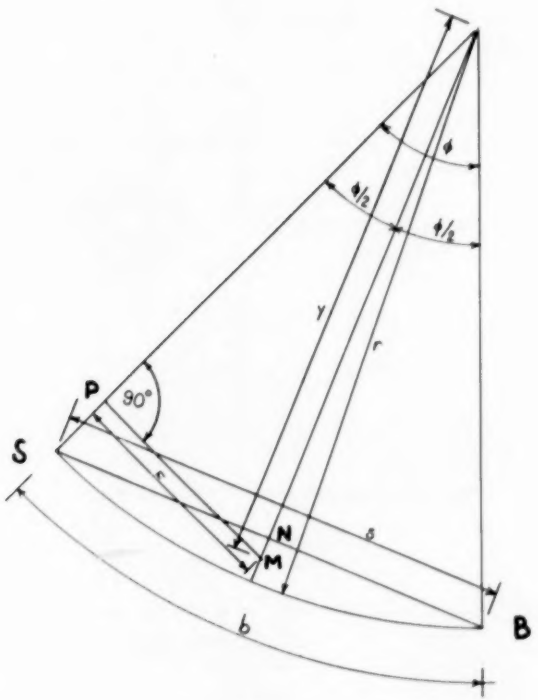


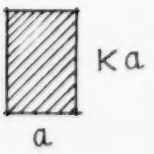
Fig. 2

Setting up an expression for the minimum work by differentiating these expressions with respect to the unknown midspan flexural moment X, adding, and setting the sum equal to zero, we obtain:

$$\int_0^{\phi_0} \frac{Mf_s}{EI} \times \frac{dMf_s}{dX} \times r\,d\phi + \int_0^{\phi_0} \frac{Mt_s}{GJ'} \times \frac{dMt_s}{dX} \times r\,d\phi = 0 \quad \ldots\ldots\ldots (16)$$

TABLE 1—VALUES OF μ

a/ka	μ
a/a	1.3000
$a/1.1a$	1.4365
$a/1.2a$	1.5860
$a/1.3a$	1.7485
$a/1.4a$	1.9240
$a/1.5a$	2.1125
$a/1.6a$	2.3140
$a/1.7a$	2.5285
$a/1.8a$	2.7560
$a/1.9a$	2.9965
$a/2.0a$	3.2500



Using Eq. 10 and 11, and performing the indicated differentiations, we obtain:

$$\frac{1}{EI}\int_0^{\phi_o} [X\cos\phi - q\,r^2\,(1-\cos\phi)]\cos\phi\,r\,d\phi +$$

$$\frac{1}{GJ'}\int_0^{\phi_o} [X\sin\phi - q\,r^2\,(\phi-\sin\phi)]\sin\phi\,r\,d\phi = 0\,. \quad \ldots\ldots(17)$$

Defining the ratio μ as $\mu = EI/GJ'$ and taking E/G equal to 2.35, we obtain $\mu = 0.65\ (1 + b^2/a^2)$. Values of this ratio are calculated for various section ratios in Table 1.

Performing the integrations of Eq. 17, simplifying, and making use of the above defined ratio μ, we obtain:

$$X = q\,r^2\left[4\left(\frac{\sin\phi\,(1+\mu) - \phi\cos\phi\,\mu}{2\,\phi\,(1+\mu) + \sin 2\phi\,(1-\mu)}\right) - 1\right] \quad \ldots\ldots(18)$$

If we take $\phi_o = 90$ deg $= \pi/2$, Eq. (18) reduces to:

$$X = q\,r^2\,(4/\pi - 1) = 0.274\,q\,r^2 \quad \ldots\ldots(19)$$

For ϕ equal to 90 deg, X is therefore independent of μ.

For convenience, designate the bracketed term of Eq. (18) as N. Values of N for three values of μ and various half-angles ϕ_o are calculated in Table 2. Using this expression, Eq. (10) and (11) can be simplified to:

$$Mf_\phi = q\,r^2\,[\cos\phi\,(N+1) - 1] \quad \ldots\ldots(20)$$

$$Mt_\phi = q\,r^2\,[\sin\phi\,(N+1) - \phi] \quad \ldots\ldots(21)$$

The angles for the inflection points may be determined by equating these expressions to zero. Then the angle for the inflection point of flexural moment becomes:

$$\phi = \text{arc cos}\ 1/(N+1) \quad \ldots\ldots(22)$$

By expanding the sine function in series form, the angle for the inflection point of torsional moment is approximately:

$$\phi, \text{ in radians} = 6N/(N+1) \quad \ldots\ldots(23)$$

TABLE 2—VALUES OF *N*

$X = q r^2 N$

ϕ_o	$\mu = 1.30$	$\mu = 2.1125$	$\mu = 3.25$
15	0.0116	0.0116	0.0116
20	0.02	0.02	0.02
25	0.0308	0.0308	0.03
30	0.044	0.0432	0.042
35	0.0584	0.0572	0.0552
40	0.0748	0.0724	0.07
45	0 0924	0.0888	0.0852
50	0 1108	0.1064	0.102
55	0.1312	0.1256	0.12
60	0.1512	0.1448	0.1388
65	0.172	0.1652	0.1588
70	0.1928	0.1860	0.18
75	0.2136	0.2072	0.202
80	0.2344	0.2292	0.2248
85	0.2544	0.2510	0.2484
90	0.2735	0.2735	0.2735

Example

Take $2\phi_o = 90$ deg, $\phi_o = 45$ deg $= 0.7852$ radians, $\mu = 1.30$.
From Table 2, $X = 0.0924\ q r^2$.
The moments (Table 3) at the ends, by Eq. (10) and (11):

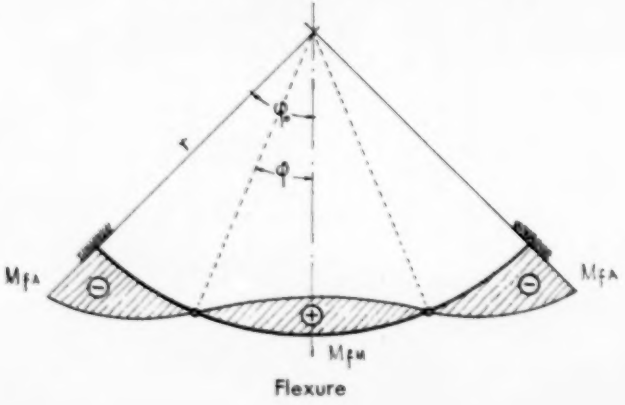
$$Mf_A = 0.0924\ q r^2 \times 0.7071 - q r^2 (1 - 0.7071) = -0.2275\ q r^2$$

$$Mt_A = 0.0924\ q r^2 \times 0.7071 - q r^2 (0.7852 - 0.7071) = -0.0128\ q r^2$$

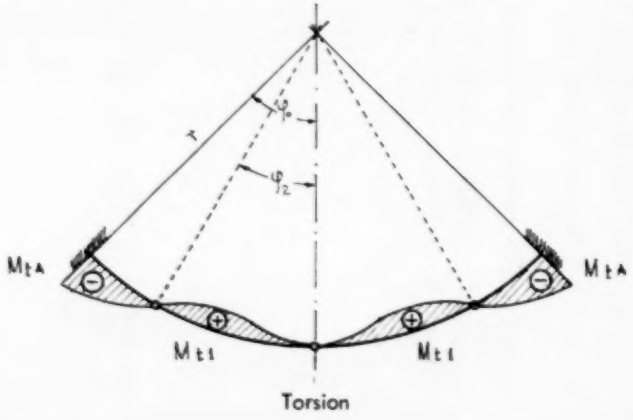
TABLE 3—VALUES OF Mf, Mt AND ϕ

ϕ_o	Mf_o	Mf_m	ϕ when $Mf = 0$	Mt_o	Mt_1	ϕ when $Mt = 0$*
			$\mu = 1.3$			
30	$-0.0959\ qr^2$	$+0.044\ qr^2$	16°42′	$-0.0015\ qr^2$	$+0.00863\ qr^2$	28°48′
45	$-0.2276\ qr^2$	$+0.0924\ qr^2$	22°48′	$-0.0128\ qr^2$	$+0.02561\ qr^2$	40°48′
65	$-0.51351\ qr^2$	$+0.1512\ qr^2$	29°42′	$-0.0909\ qr^2$	$+0.05215\ qr^2$	50°53′
75	$-0.6859\ qr^2$	$+0.2136\ qr^2$	34°30′	$-0.1365\ qr^2$	$+0.0854\ qr^2$	58°51′
90	$-1.0000\ qr^2$	$+0.274\ qr^2$	38°24′	$-0.297\ qr^2$	$+0.1209\ qr^2$	65°00′
			$\mu = 2.1125$			
30	$-0.0966\ qr^2$	$+0.0432\ qr^2$	16°33′	$-0.0019\ qr^2$	$+0.0083\ qr^2$	28°34′
45	$-0.2301\ qr^2$	$+0.0888\ qr^2$	23°18′	$-0.01535\ qr^2$	$+0.02404\ qr^2$	40°06′
65	$-0.5076\ qr^2$	$+0.1652\ qr^2$	30°00′	$-0.0782\ qr^2$	$+0.05913\ qr^2$	52°51′
75	$-0.6876\ qr^2$	$+0.2072\ qr^2$	34°06′	$-0.14272\ qr^2$	$+0.08172\ qr^2$	58°41′
90	$-1.0000\ qr^2$	$+0.2735\ qr^2$	38°24′	$-0.297\ qr^2$	$+0.1209\ qr^2$	65°00′
			$\mu = 3.25$			
30	$-0.0977\ qr^2$	$+0.0420\ qr^2$	16°18′	$-0.00214\ qr^2$	$+0.0081\ qr^2$	28°10′
45	$-0.2330\ qr^2$	$+0.0852\ qr^2$	22°52′	$-0.0179\ qr^2$	$+0.0229\ qr^2$	39°21′
65	$-0.5102\ qr^2$	$+0.1588\ qr^2$	30°12′	$-0.0890\ qr^2$	$+0.0558\ qr^2$	51°58′
75	$-0.6890\ qr^2$	$+0.2020\ qr^2$	33°42′	$-0.1477\ qr^2$	$+0.0788\ qr^2$	57°18′
90	$-1.0000\ qr^2$	$+0.2740\ qr^2$	38°42′	$-0.2970\ qr^2$	$+0.1209\ qr^2$	65°00′

*Mt goes to zero at midspan, without reversal of sign.



Flexure



Torsion

By Eq. (20) and (21) these moments are:

$$Mf_A = q\,r^2\,[0.7071\,(0.0924 + 1) - 1] = -0.2276\,q\,r^2$$

$$Mt_A = q\,r^2\,[0.7071\,(0.0924 + 1) - 0.7852] = -0.0128\,q\,r^2$$

Maximum and minimum values of Mf and Mt, together with the location of inflection points, are calculated for three values of μ and various half-angles ϕ_o in Table 3.

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Title No. 53-59

Mortar- and Concrete-Making Properties of Natural Sands Related to Their Physical Attributes*

By C. F. ZEITSMAN†

SYNOPSIS

Methods for determining mean particle size (fineness modulus), particle size distribution, particle shape, and solid content of natural sands are presented. It is shown how these properties relate to the water requirements of mortars and concretes proportioned by a recognized system.

Using ¾-in. and 1-in. crushed stone as coarse aggregate and proportioning by ACI's recommended practice for proportioning of concrete mixes, at a constant water-cement ratio, it was found that water requirements to produce concrete of a given slump could be predicted to 95 percent accuracy for 95 percent of the sands used in the experiments.

INTRODUCTION

While it is well known that fineness, grading, and shape of aggregates are important factors in concrete production, there is as yet no precise information on exactly how important these factors are. This deficiency is considered due to the absence of suitable methods for measuring and expressing grading and shape or parameters closely related thereto.

In American and British standard specifications, sand grading is stipulated by limiting the percent passing each of six sieves. In South Africa compliance with these specifications can only be achieved at considerable cost, and while it is known how much it costs in transport and labor to blend different sands to a desirable grading, it is not known what savings the desirable grading can effect.

Millions of cubic yards of concrete have been made in South Africa with sands which do not comply closely with American and British grading practices. Recently, however, urban contractors have become grading-conscious and now get large supplies of washed and graded sand from stone crushers. The South African Railways Administration has only resorted to crushed sands in cases of emergency, because in spite of good grading in crusher sands, the serviceability of crushed sand compared unfavorably with that of natural sands. The reason for this must relate to particle shape.

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In 1953 studies were begun to find out how the measurable physical attributes of South African sands affect their serviceability in concrete work. It was assumed that sand is chemically inert, sufficiently tough to resist attrition during the mixing process, and practically nonabsorbent. Thus critical factors relate to the shapes and dimensions of particles, and these factors must obviously be measured collectively and by the simplest adequate methods.

Notation

FM = fineness modulus, an expression indicative of the mean grain size of sand
σ = standard deviation of particle size, or standard scatter of grains over successive mesh sizes
SG = specific gravity
SC = solid content, ratio of actual to apparent volume of sand
SU = shape factor, derived from σ and SC by formula SU = SC $-0.09\,(\sigma - 1)$

PARTICLE SIZE

Fineness modulus, or mean particle size

Reflection shows that the well-known fineness modulus is a measure of mean particle size. It is computed from six proportions measured independently:

a = proportion greater than No. 4 sieve
b = proportion greater than No. 8 sieve
c = proportion greater than No. 16 sieve
d = proportion greater than No. 30 sieve
e = proportion greater than No. 50 sieve
f = proportion greater than No. 100 sieve

The fineness modulus is given by:

$$\text{FM} = a + b + c + d + e + f \qquad (1)$$

It follows that if all the sand passes No. 100 sieve the fineness modulus is zero. If all the sand is coarser than No. 100 but finer than No. 50, FM = 1, etc. It is observed therefore that the fineness modulus is really the weighted mean:

$$\text{FM} = 1\,(f - e) + 2\,(e - d) + 3\,(d - c) + 4\,(c - b) + 5\,(b - a) + 6a \qquad (2)$$

In this equation, $f - e$, $e - d$, etc., represent weight proportions of sand in successive intervals of particle size.

Particle size distribution

A standard deviation of particle size can be computed from the same six measurements a to f. Denoting the standard deviation by σ,

$$\begin{aligned}\sigma^2 &= (f - e) + 4\,(e - d) + 9\,(d - c) + 16\,(c - b) + 25\,(b - a) + 36a - (\text{FM})^2 \\ &= f + 3e + 5d + 7c + 9b + 11a - (\text{FM})^2 \\ &= f + e + d + c + b + a - (\text{FM})^2 + 2\,(e + 2d + 3c + 4b + 5a) \\ &= 2\,(e + 2d + 3c + 4b + 5a) - (\text{FM})\,(\text{FM} - 1) \qquad (3)\end{aligned}$$

SOLID CONTENT

Methods of measuring solid content

Three methods were employed for measuring solid content:

1. A rimless 1-gal. tin is filled with dry sand and placed on a flow table. The sand is compacted by five drops of the table of ½-in. each. More sand is put in, a further five

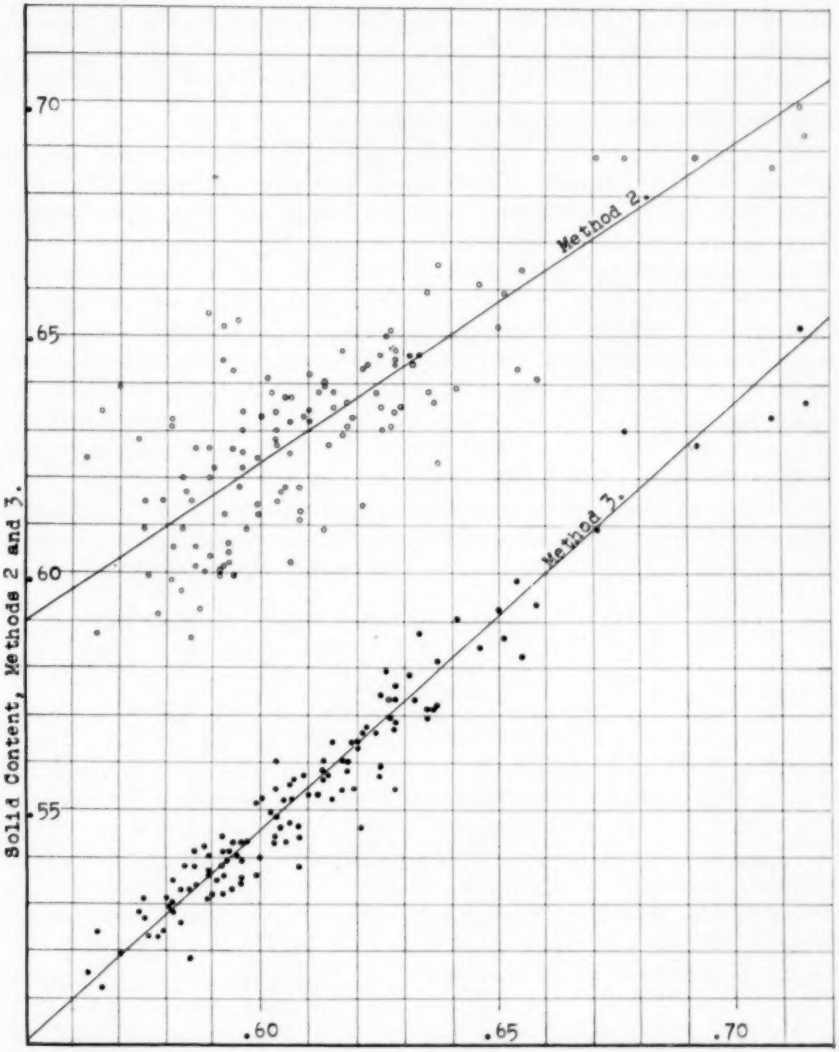


Fig. 1—Comparison of methods of determining solid content of sand

drops are given and the excess is struck off level. The actual volume of sand is found from its weight and specific gravity.

2. A 1-l measuring cylinder is half-filled with water and 1 kg of sand slowly poured into it. The flask is rotated and the contents allowed to settle until the surface is level.

3. A 300-g amount of sand is put into a cylindrical container. A free-sliding plunger weighing 1300 g is lowered onto the sand and rotated twice to level the bed. The depth of sand is measured by means of a micrometer and locating studs on cylinder and plunger.

Results obtained by the three methods are compared in Fig. 1. The dry measurements (methods 1 and 3) show the best correlation, results according to method 1 being almost exactly 10 percent higher than those of method 3.

Solid content related to particle size distribution

To find a relation between σ and solid content, the solid content of some 45 sand samples was measured. The same sands were then screened to exclusive particle sizes and the solid content measured on these. It was then possible to plot increase in solid content against increase of σ. Due to the limited quantities available method 3 had to be used, and solid content of exclusive sizes could only be done on the predominant sizes in each sample, although it was already clear that different grain sizes in one sand sample often vary appreciably in solid content and the results accordingly show a somewhat wide scatter. The results were averaged in intervals of σ and these centroids plotted in Fig. 2.

By method 3, SC increases by 0.082 as σ increases by 1. By method 1, therefore, SC increases by 0.090 as σ increases by 1.

Worthington[2] has made similar tests on coarse aggregates to determine solid content for varying gradings, but he did not use a single figure to express

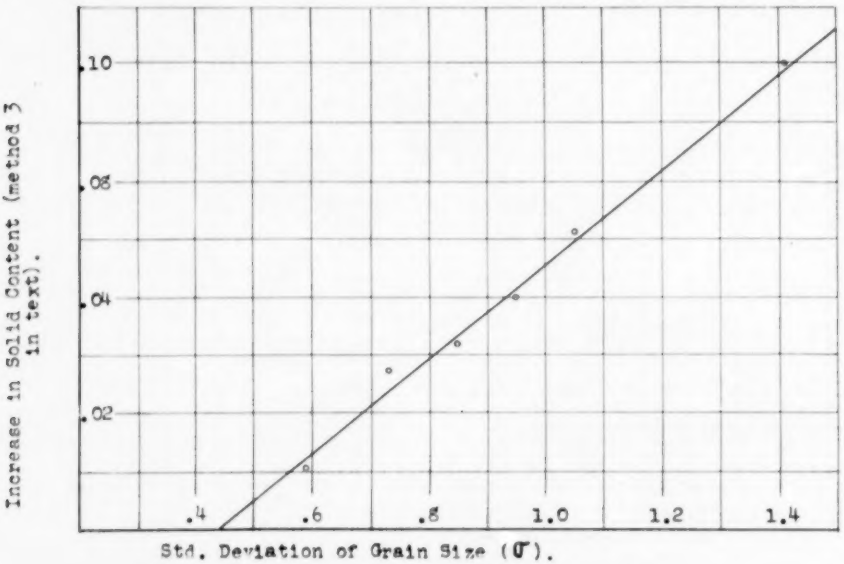


Fig. 2—Solid content as related to particle size distribution

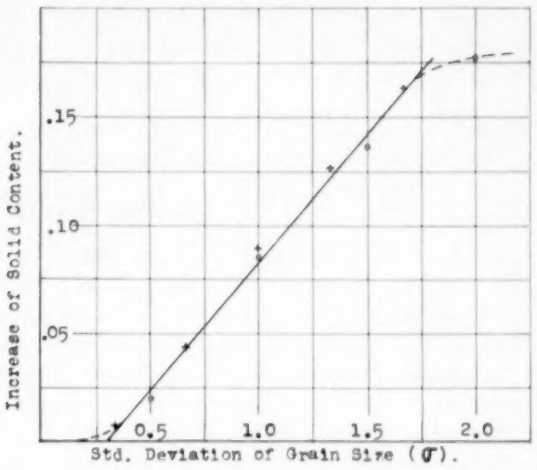


Fig. 3—Relation between σ and solid content of sand gradings. Adapted from Worthington's Fig. 5 and 7.

particle size distribution. Expressing his gradings in terms of our σ, Fig. 3 is obtained. Fig. 3 indicates an increase of 0.115 in solid content for σ increasing by 1. This higher figure is probably due to the higher degree of compaction obtained with his method of hard rodding, a method which involves the risk of excessive attrition.

For the remainder of this investigation method 1 alone was used because of its good reproducibility, and the relation between grading and solid content by this method was established as

$$\frac{\delta\, SC}{\delta\, \sigma} = 0.09 \qquad (4)$$

PARTICLE SHAPE

Evaluation of particle shape has been attempted by various methods, notably the British standard flakiness index method and the Krumbein roundness index method. These methods lack the advantage of speed. Weymouth[1] has indicated that shape could be assessed via the solid content of material screened to exclusive size.

In the present investigation, an attempt was made to establish a collective shape factor using the solid content of unscreened sand at a fixed compacting effort. It was reasoned that the solid content should be a function of particle shape and particle size distribution. Having established a simple method of evaluating the latter variable, its effect might be eliminated by means of an easily applied adjustment.

Shape factor

With the help of Eq. (4) a comparative shape factor was developed.

The mean value of σ for natural sands is approximately 1. It is therefore

convenient to define the shape factor as the solid content (method 1) of sand so graded that $\sigma = 1$. Using Eq. (4):

$$SU = SC - 0.09\,(\sigma - 1) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

where SU is the shape factor and SC the solid content of sand of particle size distribution σ when compacted by method 1.

MORTAR VALUE

Knowing the effect of σ on the quantity of sand that can be compacted into a given volume, it is anticipated that there will be some corresponding effect on the quantity of sand that can be added to unit volume of cement paste while maintaining a fixed degree of workability or flow.

Reference 3 contains data obtained on some 300 sand samples which were tested in mortar of 100 percent flow. Significant correlations were not disclosed, however, perhaps because of some gradual variation in testing technique or equipment. When some of these results were compared in chronological order, a clear increase in mortar value was seen for sands tested later in the series, although average characteristics for the first and last groups of sands tested were much the same. It is not known how much emphasis was placed on this test, but indications are that strict precautions and continual analysis may be desirable in this type of work.

Mortar value defined

For the purpose of this work, mortar value is defined as the ratio of sand to cement in a mortar of water-cement ratio 0.6 and with 100 percent flow when tested according to ASTM C 87-47[4]. If the sand has a specific gravity of 2.65 the ratio is given by weight; otherwise a small correction is made. For example, if the SG of the sand is 2.70 the observed mortar value is multiplied by 2.65/2.70.

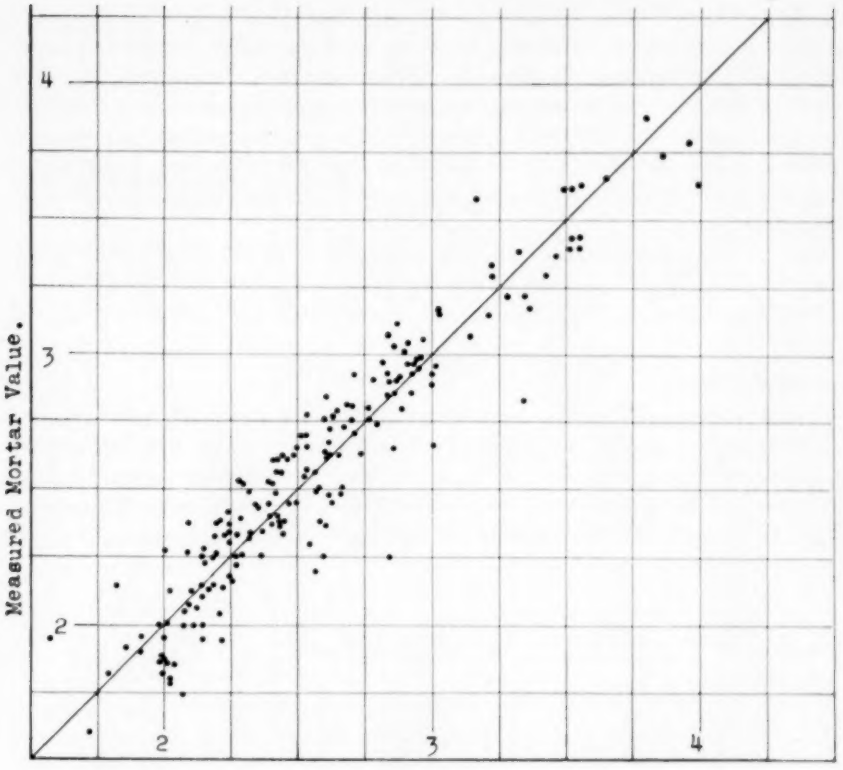
Test results

Mortar values were measured for some 210 sand samples with fineness moduli ranging from 1 to 4, σ from 0 to 1.6, and SU from 0.58 to 0.71. Sands were sub-sampled by means of a riffle sampler. From time to time some of the first samples were retested to insure that there was no unknown disturbing factor of any significance.

In the analysis it was found that results became spurious when more than 10 percent of the sand passed the No. 100 sieve. Since such sands will seldom be tolerated in a specification they were omitted in the final analysis. By graphical means the following equation was derived from the remaining 185 results:

$$MV = 2.16 + \tfrac{1}{2}\,\sigma\,(FM - 1.5) + 12.5\,(SU - 0.62) \quad \ldots\ldots\ldots\ldots (6)$$

In Fig. 4 the experimentally determined mortar values are plotted against mortar values computed from Eq. (6). It appears that over 90 percent of the estimated values differ from the observed values by less than 0.25.



Mortar Value calculated from equation 6.

Fig 4.—Calculated mortar value compared with measured mortar value

CONCRETE-MAKING VALUE

The mortar-making value is associated with the amount of fine aggregate that can be mixed with a unit amount of cement paste, without exceeding a given flow. By similar reasoning the concrete-making value is associated with the amount of combined aggregate that can be mixed with a unit amount of cement paste without exceeding a given slump. In both cases a maximum workability with a minimum of water is desired. With two aggregates involved the problem is much more complicated because the best slump also depends on the best ratio of fine to coarse aggregate, and much more time and material are necessary for a test.

Proportioning the mix

When this investigation was planned, the South African Railways Administration was using an adapted version of ACI 613-44[5] as a basis for concrete mix proportioning. With coarse aggregates smaller than 1½ in. this method

usually involves a larger proportion of sand than commonly used in British practice, but mixes were found to be more workable and less prone to segregation and honeycombing in the field. It had been found that a small decrease in the sand-stone ratio sometimes resulted in a slightly better slump, but the gain is somewhat doubtful as it generally involves either greater risk of segregation and greater variability of crushing strength or greater demands on labor and supervision, while this administration gets sand appreciably cheaper than stone.

To facilitate investigation on a large selection of sands with limited laboratory facilities, it was decided to use one type of stone of controlled size only and to proportion sand and stone as recommended for trial mixes in ACI 613-44[5].

Preliminary tests

In a series of rapid preliminary tests, mixes were deliberately oversanded to give the varying sand characteristics a good chance of asserting themselves. To do this on some systematic basis, mixes were proportioned as for ½-in. to ¾-in. stone, while the actual stone used was ¾ in. to 1 in. Each mix consisted of 2.2 kg cement, 1.3 kg water, and 10.8 kg combined aggregate, with the combined aggregate as follows:

Fineness modulus of sand	Quantity of sand, kg	Quantity of stone, kg
Under 1.60	4.8	6.0
1.60 to 1.99	5.0	5.8
2.00 to 2.39	5.2	5.6
2.40 to 2.79	5.4	5.4
2.80 to 3.19	5.6	5.2
3.20 to 3.59	5.8	5.0
3.60 and over	6.0	4.8

Results of these preliminary tests showed clearly that slump increased as grading and shape of sand improved. Unfortunately many sands gave slumps outside the 1- to 7-in. range, where the slump test is not quantitatively discriminating.

A further series of tests with mixes proportioned as before, but using ½-in. to ¾ in. stone, confirmed the trend. Comparing results for similar sands, it appeared that about 1 in. slump is gained by using the larger stone, which agrees closely with the figure derived from ACI 613-44, although a higher sand proportion was used with the 1-in. stone than recommended.

In the preliminary tests the fineness modulus *per se* did not seem to affect the slump.

Planning further tests

It had become clear from the preliminary tests that simplification of sand-stone ratio had no drastic effect, and that measuring slumps at constant water content was not adequate due to the limitations of the slump test.

To overcome the latter handicap it was decided to vary water content (as is done in the ASTM standard flow test for mortar) until a slump of about 4 in. was obtained. As in the ASTM mortar test, the water-cement ratio had to remain constant, an increase in water being accompanied by a proportional increase in cement.

Effect of water content on slump

The effect of water content on slump was established by tests on some 50 sand samples. In each case the proportion of cement paste (having constant W/C ratio) was varied to produce a range of slumps. An empirical equation was established, based on response of slump to water changes.

$$x = 0.6 + \tfrac{1}{2}\left(\frac{w}{16.85} - c\right)^2 \qquad (7)$$

where x = slump in in.
w = water, lb per cu yd concrete
c = constant for a particular sand

It must be emphasised that Eq. (7) holds for concrete made of ½-in. to ¾-in. stone and natural sand, proportioned by ACI 613-44. Extension to other conditions will need further proof.

From Eq. (7) the water requirement for 4-in. slump, say, can be readily derived from the observed water requirement giving any slump between 1½ and 7½ in. Table 1 has been compiled to facilitate such adjustments.

TABLE 1—AID IN DERIVING WATER REQUIREMENT FOR 4-IN. SLUMP (W_4) BY EQ. (7) WHEN WATER REQUIREMENT AT SLUMP x IS KNOWN

x, slump, in.	$W_4 - W_x$ lb per cu yd	x, slump, in.	$W_x - W_4$, lb per cu yd
1.0	28.8	4.25	1.5
1.25	24.8	4.5	3.0
1.5	21.4	4.75	4.5
1.75	18.4	5.0	6.1
2.0	15.8	5.25	7.4
2.25	13.3	5.5	8.8
2.5	11.1	5.75	10.1
2.75	9.1	6.0	11.5
3.0	7.1	6.25	12.6
3.25	5.2	6.5	14.0
3.5	3.4	6.75	15.2
3.75	1.7	7.0	16.3
		7.25	17.5
		7.5	18.5

*These corrections are derived from empirical Eq. (7) which solved for w gives: $w = 16.85\,[c + \sqrt{2\,(x - 0.6)}]$.

Example: If 340 lb water per cu yd concrete gives 2¼-in. slump, how much water is required for 4-in. slump?

$$340 = 16.85\,[c + \sqrt{2(2.25 - 0.6)}]$$

$$W_4 = 16.85\,[c + \sqrt{2\,(4 - 0.6)}]$$

$$W_4 - 340 = 16.85\,[\sqrt{0.8} - \sqrt{3.3}\,] = 13.33$$

$$W_4 = 340 + 13.3 = 353 \text{ lb}$$

TABLE 2—DERIVED VALUE OF W_4 FOR SANDS BASED ON FOUR INDEPENDENT OBSERVATIONS OF SLUMP VARYING FROM 1 TO 7½ IN.

Sand sample No.	W_4, lb per cu yd of concrete for 4-in. slump				Average, lb	Range, lb
1	374	373	376	373	374	3
2	425	428	431	421	426	10
3	366	367	367	367	367	1
4	397	395	393	393	394	4
5	370	370	370	373	371	3
6	381	384	385	385	384	4
7	351	350	346	356	351	10
8	404	407	408	406	406	4
9	363	367	364	363	364	4
10	399	399	396	396	397	3
11	381	383	383	387	383	6
12	387	388	387	386	387	2
13	397	396	399	396	397	3
14	389	392	388	392	390	4
15	408	405	408	409	407	4
16	354	368	362	364	362	14
17	376	374	374	371	374	5
18	304	312	314	318	312	14
19	339	338	340	345	340	7
20	347	346	346	345	346	2
21	357	353	350	347	352	10
22	355	357	361	367	360	12
23	387	393	396	396	393	9
				Average:	375.5	6.0

To check the validity of this statement, Table 2 shows the derived water requirements for 4-in. slump for 23 sands. On each of these sands four slump tests were made, slumps varying from 1 to 7½ in.

Insofar as the derived water requirement is based on actual slump tests, it is for further purposes referred to as "observed water requirement for 4-in. slump" and designated "W_4 observed." "W_4 estimated" refers to estimates based on measurements on the dry sand alone.

Table 3 shows typical test results on concrete made with ½- to ¾-in. stone and mixes proportioned according to ACI 613-44; this table represents about one-fifth of the total tests conducted.* Estimated water requirement was based on Eq. (10) derived later in this paper.

ANALYSIS OF TEST RESULTS

Having found from the preliminary tests that fineness modulus of sand, if properly compensated for in the sand-stone ratio, is not a dominant factor, the results were first analysed with special attention to SU (shape factor) and σ. Results were grouped into classes according to σ (grain spread) and water requirement plotted against SU for each class. A family of roughly parallel curves was obtained. Increasing σ by 0.1 had the same effect on water requirement as increasing SU by 0.006, that is, the water requirement is a function of J where

$$J = \text{SU} + 0.06\,(\sigma - 1) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Referring to Eq. (5), SU is defined as SC − 0.09 (σ − 1); therefore

$$J = \text{SC} - 0.03\,(\sigma - 1) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

*Full list of test results is available from the Chief Civil Engineer's Office, South African Railways Administration, Johannesburg, South Africa.

TABLE 3—CONCRETE-MAKING PROPERTIES. TYPICAL RESULTS FROM STUDY OF NATURAL SANDS*

Percentage retained by Tyler sieve No.						FM	σ	SG	SC	SU	Water, lb per cu yd concrete: ¾-in. maximum stone, 4-in. slump	
4	8	14	28	48	100						Observed	Estimated*
2.4	6.8	20.5	49.7	81.0	94.3	2.55	1.30	2.63	0.652	0.625	347	345
	1.9	12.5	21.9	59.6	93.6	1.90	1.13	2.68	0.668	0.656	326	329
1.0	1.8	5.1	21.3	82.9	96.2	2.08	0.90	2.67	0.610	0.619	387	382
2.4	4.0	6.6	13.0	58.1	91.0	1.75	1.15	2.65	0.621	0.608	390	391
		5.8	46.2	90.9	98.5	2.41	0.77	2.64	0.616	0.637	375	368
		1.1	38.8	90.6	98.9	2.29	0.68	2.64	0.596	0.625	386	390
		1.5	30.2	94.0	99.7	2.25	0.59	2.65	0.595	0.632	375	389
		2.0	3.7	69.0	93.3	1.66	0.66	2.65	0.595	0.626	388	405
			1.9	64.6	94.1	1.61	0.63	2.65	0.599	0.632	394	399
0.2	0.7	6.1	43.7	87.6	98.9	2.37	0.82	2.64	0.616	0.632	371	370
		6.1	68.6	68.3	96.7	2.58	0.88	2.65	0.683	0.695	308	316
			22.1	61.5	92.3	1.76	0.88	2.64	0.665	0.676	360	340
1.9	3.9	28.5	80.7	98.6	99.8	3.13	0.85	2.64	0.613	0.627	350	375
1.0	9.0	41.5	83.4	99.0	99.9	3.34	0.91	2.63	0.617	0.625	367	372
2.1	4.2	12.7	52.3	91.0	97.4	2.60	1.03	2.67	0.633	0.630	355	356
0.8	1.6	4.3	19.7	88.5	99.0	2.14	0.77	2.66	0.609	0.630	385	378
	1.0	3.5	16.8	85.1	97.8	2.04	0.73	2.66	0.603	0.627	391	384
2.1	8.7	25.7	57.0	88.4	97.9	2.80	1.21	2.62	0.635	0.616	358	360
			4.8	66.6	98.3	1.61	0.74	2.64	0.609	0.632	373	391
0.1	0.1	0.3	40.8	95.8	99.8	2.37	0.58	2.65	0.615	0.653	367	362
0.3	0.9	12.8	77.9	98.4	99.9	2.90	0.67	2.65	0.623	0.653	348	355
1.0	4.3	14.0	51.7	91.1	99.5	2.62	0.97	2.64	0.644	0.647	355	344
1.0	6.2	33.0	73.0	97.2	99.6	3.10	0.96	2.62	0.651	0.619	373	377
0.6	11.9	41.2	65.9	82.1	93.2	2.96	1.42	2.64	0.711	0.673	313	312

*In the entire test program there were 22 sands with FM between 1.37 and 1.84. On average, observed values for these are 14 lb higher than the estimated values; estimated values shown in this table have been adjusted accordingly. Standard error for these 22 sands is 13.5 lb, but this is of little practical import since such fine sands are not permitted in concrete work.

In the next stage the observed water requirement was plotted against J and a curve drawn. It was found that fineness modulus had practically no effect on water requirement within the normally accepted range, but that water demand increased steeply for fineness modulus less than 1.85.

Deleting sands of fineness modulus less than 1.85, Eq. (10) was developed.

$$\left[\frac{W_s}{10} - 37.2\right] + 9 = \sqrt{9^2 + 13.2^2 - (13.2 - [100J - 62])^2} \quad \ldots\ldots\ldots (10)$$

Table 4 is given to facilitate solving Eq. (10).

Fig. 5 shows observed water demand plotted against water demand estimated from Eq. (10) for natural sands of fineness modulus from 1.85 to 3.8. The standard error of estimate is 8.3 lb water per cu yd of concrete for the 101 sands in this range of fineness modulus.

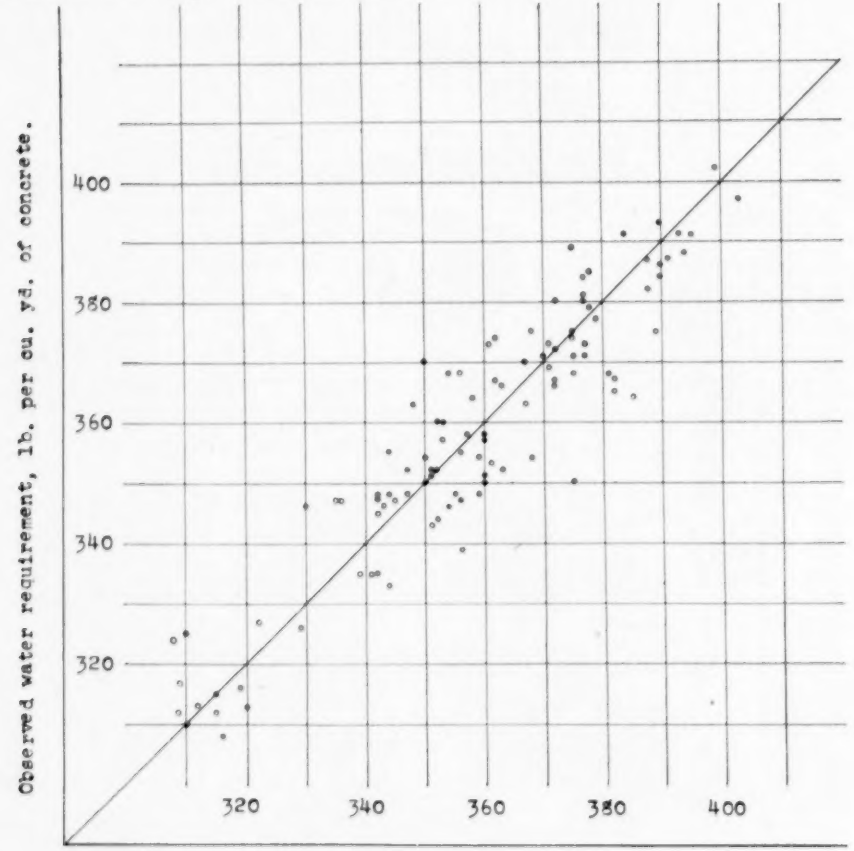


Fig. 5—Theoretical versus observed water requirement for natural sands with FM from 1.85 to 3.8.

TABLE 4—WATER REQUIREMENT, LB PER CU YD OF CONCRETE, TO PRODUCE 4-IN. SLUMP USING ½- TO ¾-IN. CRUSHED STONE AND NATURAL SAND OF THE INDICATED *J*-VALUE, BASED ON EQ. (10)*

J	.000	.001	.002	.003	.004	.005	.006	.007	.008	.009
0.57	419	418	418	417	417	416	415	415	414	413
0.58	413	412	411	410	410	409	408	407	407	406
0.59	405	404	403	402	402	401	400	399	398	397
0.60	396	395	394	393	392	391	390	389	388	386
0.61	385	384	383	382	380	379	378	376	375	374
0.62	372	371	369	368	366	365	364	363	361	360
0.63	359	358	357	355	354	353	352	351	350	349
0.64	348	347	346	345	344	343	342	342	341	340
0.65	339	338	337	336	336	335	334	334	333	332
0.66	331	330	330	329	329	328	327	327	326	326
0.67	325	324	324	323	323	322	322	321	320	320
0.68	319	319	318	318	317	317	316	316	316	315
0.69	315	314	314	313	313	313	312	312	312	311
0.70	311	311	310	310	310	309	309	309	308	308
0.71	308	308	307	307	307	307	306	306	306	306
0.72	306	305	305	305	305	305	304	304	304	304

*NOTE—This table is a solution of Eq. (10) for W_4, with values of J ranging from 0.570 to 0.723. Use both the top row of numbers and the left hand column to establish J to three significant figures.
For sand of fineness modulus less than 1.85 the water requirement is higher and highly variable. For ¾- to 1-in. stone the water requirement is some 3 percent less.

Reanalysis of preliminary test results

Although preliminary results with ¾-in. to 1-in. stone were inadequate for developing a formula, they may be compared with Eq. (10). Typical details necessary for such comparison are given in Table 5. Estimates were derived from Eq. (10) and then decreased 3 percent to allow for the larger stone.

Deleting sands of fineness modulus less than 1.85 as before, 54 results were left for comparison. Standard error of estimate for these 54 sands was 9.8 lb. It is reasonable to expect a higher standard error in these preliminary tests. Observed slumps were supported by one test per sand only, and a large adjustment was involved in deriving water requirement for 4-in. slump from water requirement for 1-in. slump.

MANUFACTURED SANDS

Crusher sands do not conform accurately to Eq. (10). From Fig. 6 it is clear that a larger error of estimate is involved, nearly 15 lb as compared to 8.3 lb for natural sands. Sands containing more than 12 percent material finer than the No. 100 sieve were deleted. It was observed that thoroughly washed sands are much more predictable than sands which have been merely moistened during manufacture—a common practice in South Africa. It is suspected that the flaky dust may be the aggravating factor. While it is possible that dust flakes can nestle together in the dry test for solid content, thus denying the really bad shape, they are likely to disperse when mixed in concrete, causing extreme harshness and excessive water demand.

It was found that slump decreased rapidly as the percentage of material finer than No. 100 sieve increased. With natural sands and well washed crushed sands the problem is not as bad. It was concluded that the finer the particle the more important the shape as far as workability is concerned.

TABLE 5—TYPICAL DETAILS OF REANALYSIS OF PRELIMINARY TEST RESULTS*

FM	σ	SC	SU	Water, percent	Water, lb per cu yd concrete, 1-in. maximum stone, 4-in. slump	
					Observed	Estimated
2.40	1.40	0.631	0.595	21.27	358	362
2.83	1.09	0.629	0.621	21.01	354	353
2.80	1.14	0.629	0.616	21.01	354	354
2.68	0.70	0.601	0.628	22.15	373	373
2.15	0.58	0.587	0.625	22.89	386	384
2.37	0.76	0.602	0.624	22.56	380	374
1.94	0.61	0.591	0.626	22.90	386	381
2.43	0.65	0.592	0.624	23.08	389	380
2.91	0.72	0.596	0.621	21.72	366	380
2.10	0.72	0.592	0.617	23.33	373	384
2.41	0.75	0.594	0.623	23.32	372	382
2.27	0.68	0.591	0.620	23.33	393	383
2.19	0.95	0.603	0.608	21.70	366	379
2.14	1.05	0.609	0.604	21.75	366	377
2.32	1.24	0.603	0.581	21.96	370	388
2.06	1.14	0.595	0.582	22.66	382	392
2.19	1.15	0.623	0.609	22.25	375	364
1.90	0.69	0.592	0.620	23.02	388	383
2.50	1.28	0.619	0.594	21.69	365	372
2.12	0.77	0.591	0.612	22.74	382	386
2.68	0.66	0.620	0.651	20.67	348	348
2.78	0.77	0.628	0.649	21.12	356	342
2.39	1.00	0.620	0.620	21.72	366	361
2.94	1.05	0.614	0.610	20.78	350	372
2.29	0.90	0.610	0.619	21.53	368	371
2.86	0.91	0.604	0.612	21.97	370	377
2.69	0.85	0.606	0.620	22.16	373	372
2.66	0.82	0.595	0.611	22.48	379	384
2.66	0.83	0.596	0.611	22.71	382	383
2.68	1.14	0.623	0.610	21.35	359	363

*Estimated water requirements were based on Eq. (10), then decreased 3 percent to allow for the larger stone. Complete record of all sands included in these tests is available from Chief Civil Engineer's Office, South African Railways Administration, Johannesburg, South Africa.

Washing has the effect of removing essentially particles of small weight and large surface and its beneficial effect is not fully reflected by sieve analysis.

Surface texture is another possible disturbing factor in crusher sands. Etched surfaces as found in natural sands require more water for surface wetting than smooth cleavage faces as are often found in crusher sands.

It will be realized that Eq. (10) is an empirical equation, and the foregoing reflections are meant to show that there are physical reasons for classing manufactured sands in a different category. Designs should not be based on Eq. (10) alone, but actual performance tests should be carried out to confirm the estimates from time to time.

Further studies may isolate and bring under control the sources of disturbance; in the meantime Eq. (10) will only apply to natural sands.

SHAPE FACTOR—PRACTICAL VALUE

Why should the shape factor of a sand be known while Eq. (9) does not use it? The shape of sand from one source is fairly constant. Once the shape factor is known, a sieve analysis alone is sufficient to permit adjustment for

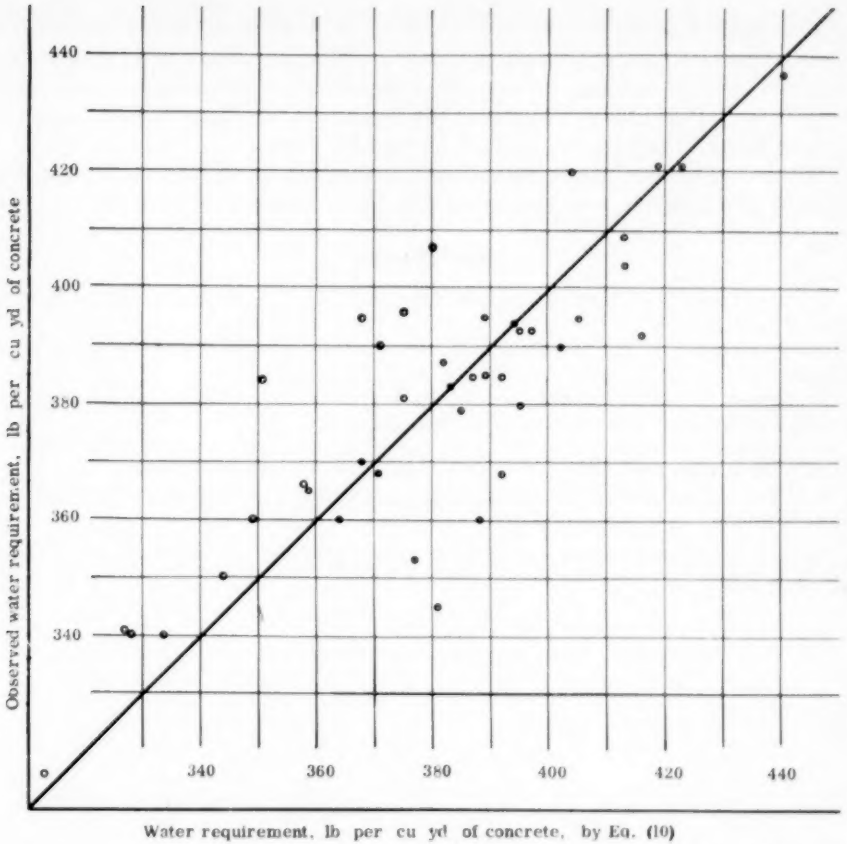


Fig. 6—Theoretical versus observed water requirement for manufactured (crusher) sands

changes in grading, and Eq. (8) will be used. The comparative shape factor is essentially a matter for a central laboratory where mechanical control can be maintained.

Given two sands of similar grain shape, the best blend can be calculated

$$FM_b = \alpha_1 FM_1 + \alpha_2 FM_2 \quad \ldots\ldots (11)$$

$$\sigma_b^2 = \alpha_1 \sigma_1^2 + \alpha_2 \sigma_2^2 + \alpha_1 \alpha_2 (FM_1 - FM_2)^2 \quad \ldots\ldots (12)$$

$$\alpha_1 \text{ (optimum)} = \tfrac{1}{2}\left[1 + \frac{(\sigma_1 - \sigma_2)(\sigma_1 + \sigma_2)}{(FM_1 - FM_2)^2}\right] \quad \ldots\ldots (13)$$

Where FM_b, FM_1, and FM_2 represent fineness modulus of the blend and its two components, respectively. σ_b, σ_1, and σ_2 are particle size distribution of blend and components, respectively. α_1 and α_2 are proportions of the two sands in the blend.

From any number of available sands, it is possible to determine from a knowledge of shape and grading which sand or combination of sands will produce the most economical concrete. Experimental elimination is so laborious that it is seldom applied, and the results are usually uncertain because it is difficult to determine the best blend with samples of limited size.

Where sands of considerably different shapes are to be blended, an experimental check on the performance of the blend should be made.

REFERENCES

1. Weymouth, C. A. G., "Designing Workable Concrete," *Engineering News-Record*, V. 121, Dec. 1938, p. 818.

2. Worthington, P. M., "An Experiment on the Theory of Voids in Granular Materials," *Magazine of Concrete Research*, V. 4, No. 12, Apr. 1953, pp 121-126.

3. Mather, Bryant, "Tests of Fine Aggregate for Organic Impurities and Compressive Strength in Mortars," *ASTM Bulletin*, No. 178, Dec. 1951.

4. "Standard Method of Test for Measuring Mortar-Making Properties of Fine Aggregate," ASTM C 87-47.*

5. "Recommended Practice for the Design of Concrete Mixes (ACI 613-44)," ACI JOURNAL, June 1945, *Proc.* V. 41, pp. 651-672.*

*NOTE—Reference 4 has been superseded by ASTM C 87-52. ACI 613-44 has been superseded by "Recommended Practice for Selecting Proportions for Concrete (ACI-613-54)," ACI JOURNAL, Sept. 1954, *Proc.* V. 51, pp. 49-64.—EDITOR.

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Title No. 53-60

An Idea for Long-Span Prestressed Concrete Bridges*

By E. VAN WALSUM†

SYNOPSIS

An imaginative solution applies prestressing to hyperboloidical and hyperbolic paraboloidical girders for long-span bridges. Problem is treated in a general, nonmathematical form.

INTRODUCTION

Continuous prestressed concrete girder bridges with spans of 200 ft or more are no longer exceptional. Several striking structures have been built during the last 10 years and each of them is of great interest because of novel solutions for new and old problems.

These problems arise from six basic requirements: (1) simple formwork; (2) easy placement of concrete; (3) high quality concrete, especially in the end blocks; (4) easy placement of steel; (5) simple tensioning and anchorage procedures; (6) elimination, or reduction to reasonable limits, of friction losses in the prestressing steel. Up to now some five different ways have been developed to meet some of these requirements.

European practice

Freyssinet and Guyon sought the solution to these problems in precasting.[1,2] Their bridges are extremely graceful in appearance, but the amount of refined detail work involved in all stages of construction is probably the reason why

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this type of bridge has up to now been confined to a series of reconstruction jobs on the Marne River.

Another good example of refined French craftsmanship is the bridge over the Seine River at Villeneuve-Saint Georges.[3,4] This bridge was cast in place. Its main feature is a truss-like arrangement of the prestressing reinforcement, using rocker bents to eliminate stress losses in the cables due to friction. Here again appear the refined details that will make this type of bridge too costly for widespread use.

More long-span concrete bridges are built in Germany than elsewhere, probably due to the stimulating effect of strong competition. The most spectacular method of bridge building there has been developed by the contracting firm of Dyckerhoff and Widmann[5] using what may be called "sliding forms" in a horizontal direction. In this way several giant cantilevered structures were built without the use of centering.* In this type of structure a maximum of formwork efficiency is combined with a sacrifice of economy in the placement and stressing of the steel, since all prestressing elements have to be spliced every 10 ft.

The Baur-Leonhardt system[6] by contrast places its main emphasis on the economy of stressing operations. All prestressing steel is concentrated in a few heavy cables which are all stressed in one operation.

Long-span prestressing in the Americas

The lead in the development of long-span prestressed concrete structures in the Western Hemisphere has been taken by Saenz-Cancio-Martin, Engineers, of Havana, Cuba, with the completion of the Cuyaguateje River Bridge, a 300-ft structure on the Pan-American Highway in Cuba.[7] This bridge may well be regarded as a forerunner of the American type of prestressed concrete bridge since it is basically simple and open to improvements. Main feature is maximum simplicity in placing and stressing of the main reinforcement which is placed inside the box girder, following the bending tensile zone, and in the top slab over the continuous supports. Since the prestressing cables are not bonded to concrete, it is possible to check the influence of creep and shrinkage on prestressing forces. This is of great value in this stage of development since it will supply engineers with reliable data on creep and shrinkage obtained from true-to-scale structures.

In future structures, however, it will be preferable to bond the steel to the concrete since it has been proved that this, under ultimate loading conditions, favorably influences the behaviour of prestressed structures, particularly in regard to deflection.

Because prestressing cables are restricted to the bending tensile zones, reinforcement of structural grade steel is still required to resist the large shear forces.

*See cover photograph of February 1957 ACI "News Letter" for a view of one of these bridges during construction.—Editor

Comparison of present systems

Relative merits of these different systems have been discussed in several articles, with discussion centering on two main points—bond between prestressing steel and the surrounding concrete, and the concentration of prestressing forces.

Designers generally agree that well-bonded steel, evenly distributed in the tensile zone, is from a technical point of view preferable to unbonded steel. For reasons of economy, however, it has often been found necessary to deviate from this rule; the question then becomes how to counteract the disadvantages.

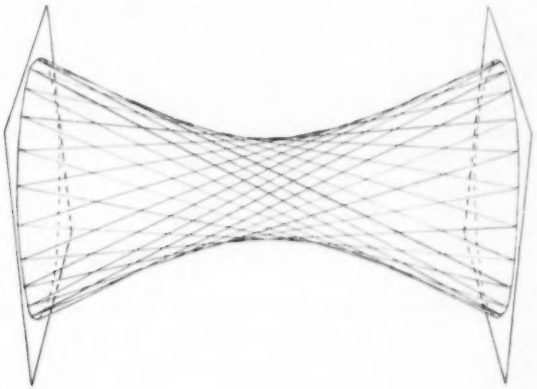

Fig. 1—Hyperboloid

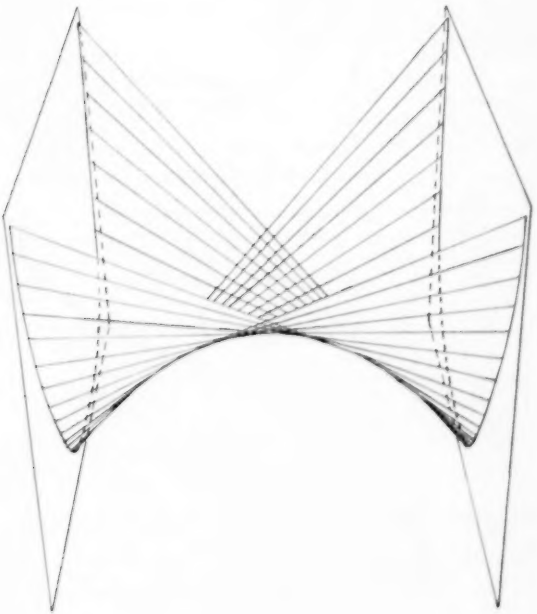

Fig. 2—Hyperbolic paraboloid

Each party involved seems to have a satisfactory solution for its technical problems, thus leaving to the contractors the decision as to which system is most economical.

In each of the five systems mentioned, designers appear to have concentrated on only some of the six basic requirements herein outlined. The author has tried to develop a bridge type which, by virtue of its shape, meets all six requirements as fully as possible.

OPTIMUM CROSS SECTION

If one compares the different shapes possible, the hyperboloid (Fig. 1) and the hyperbolic paraboloid (Fig. 2) seem to best meet the six basic requirements. Each of these two curved surfaces may be completely formed by straight lines. By exploiting this striking feature it is possible to meet all the basic requirements, while using any system of linear prestressing. In the following pages, the term "hyppar" will be used when referring to either the hyperboloid or the hyperbolic paraboloid.

THE HYPPAR BRIDGE

The external appearance of a hyppar bridge is shown in the sketch at the beginning of this paper, and in the detail of Fig. 3. Basically this is the same type of bridge as the Cuyaguateje River Bridge[7], a concrete box girder in which the main reinforcement consists of cables placed inside the box following the bending tensile zone and in the top slab over the continuous supports. Some additional features are:

1. All main reinforcement is straight so that no diaphragms are required to keep the reinforcement in the correct position and no appreciable stress losses occur in the cables due to friction.

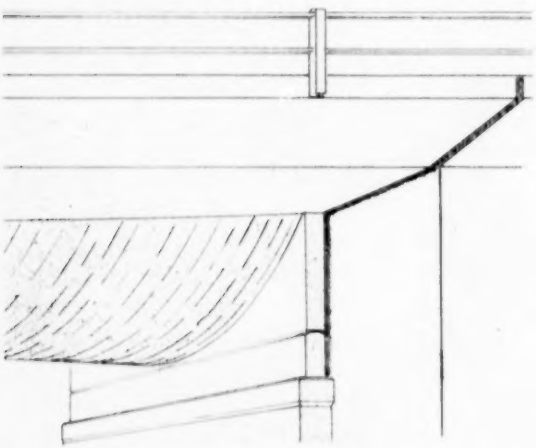

Fig. 3—Detail of hyppar bridge at abutment

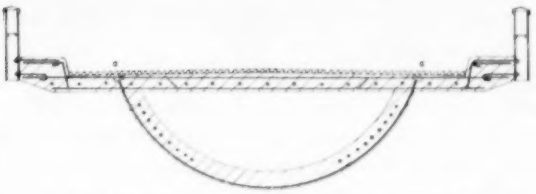

Fig. 4—Typical cross section of hyppar bridge

2. Shear reinforcement consists of prestressing cables which are easily placed and tensioned. The prestressed cables at right angles replace conventional shear reinforcement by providing compression to cancel the diagonal tension due to shear.

3. It is only for the relatively small transverse moments and shears such as occur in the deck slab, that regular reinforcement has to be used.

4. All cables are bonded to the concrete body and distributed equally through the tensile zones. Cables may be of any of the usual post-tensioned types, grouted for bond.

In this continuous box girder the underside of the "box" is curved as is shown in the cross section (Fig. 4). This curve is a segment of either a circle or a parabola, depending on whether the hyperboloid or the hyperbolic paraboloid is used. The curved part of the box is cast in place, while a partially precast deck is tied to the curved part by cables.

Formwork

To form these curved surfaces, there is one detail which requires attention and craftsmanship—the fabrication of curved supports (Fig. 5). These curved supports are either congruent parabolas or segments of a circle with varying radius. To facilitate fabrication, the congruent parabolas are preferable. Once these supports are ready, unskilled labor will be able to finish the bridge.

Fig. 6 indicates how straight members are placed over the supports completing the hyppar shape. Starting at the piers, this hyppar shape is lined with sheets of ¼-in. plywood overlapping each other and making the total plywood

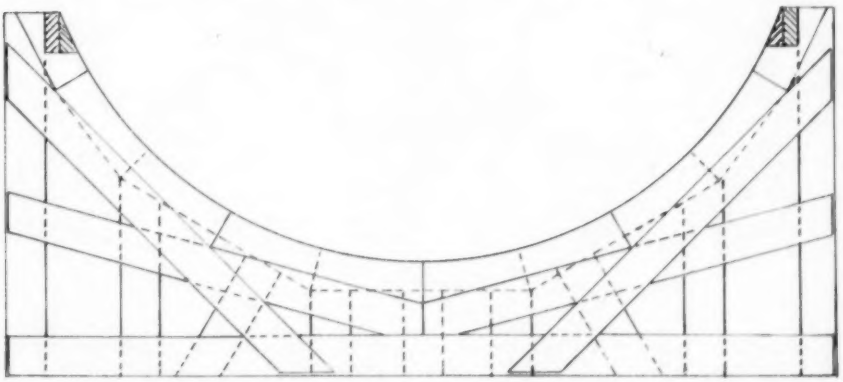

Fig. 5—Curved support for formwork

thickness a minimum of ½ in. Once the outside form is completed the prestressing cables can be placed.

Whether or not inside forms are required will depend on the way in which the concrete is placed. No inner forms may be required if a stiff mix is used. In most cases, however, it will be necessary to at least partially form the inside. The same principles as described for the fabrication of the outside forms may be applied for the inner forms. Inside and outside forms can be spaced by means of "snap-in-ties." A cross section of such a completed form is shown in Fig. 7.

Main reinforcement

In Fig. 8 are shown several straight lines, all of which lie in the surface of a hyperboloid, extending straight from one side to the other. If these lines represented prestressing cables, such cables would all be straight, easy to place, easy to tension without friction losses. If cables were placed along the right lines, then the resultant of these cable forces would have a varying eccentricity with regard to the neutral axis of the different sections, maximum at the center of the span. If reinforcement at the supports is completed with straight cables in the top slab, the whole cable arrangement for main reinforcing is nearly ideal, since the cables are evenly spread all through the tensile zones and since the cables are straight. The straightness of the cables means that they may be tensioned from one side, amounting to a 50 percent saving in tensioning costs as compared to the usual way of tensioning a cable from both sides. The non-

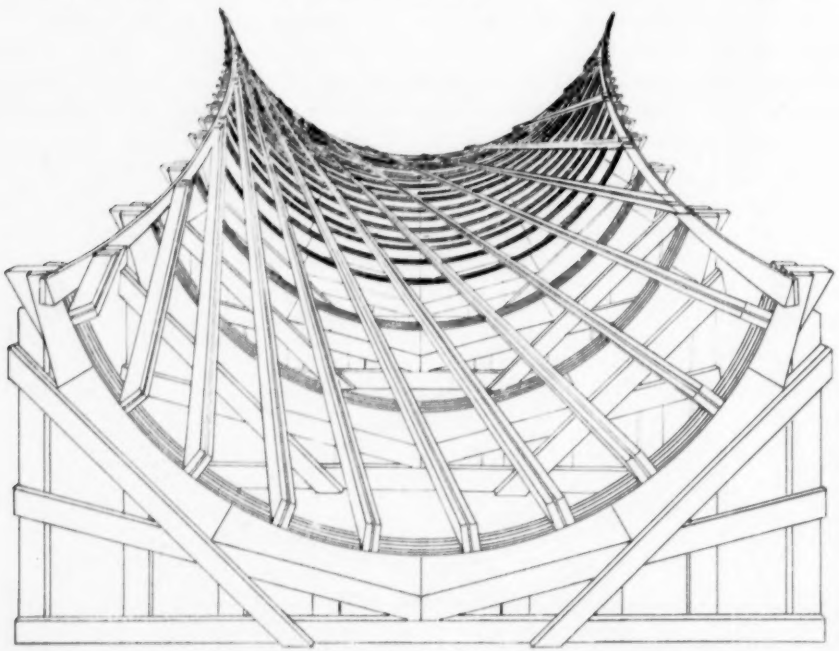

Fig. 6—Straight members, placed on the curved supports complete the hyppar shape

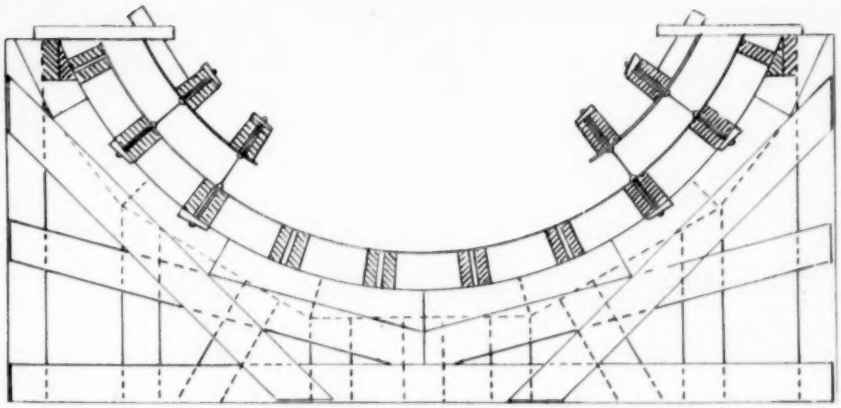

Fig. 7—Cross section of the completed form

tensioned ends of the cables are anchored within the concrete body. The ends to be tensioned of cables of adjacent spans pass each other crosswise above the interior supports so that room for placing the tensioning jack becomes available (Fig. 9). All cables are to be post-tensioned and placed in a metal sheath. Elimination of friction losses makes the scheme also attractive from a designer's point of view, since it gives more certainty that the specified prestress will be where it is intended.

Shear reinforcement

Shear reinforcement consists of cables wrapped around the curved part of the box (Fig. 4). These cables are stressed from both sides at points *a*. Their effectiveness is not reduced by friction, since the full prestress is still available where it is required, i.e. near the neutral axis where principal tensile stresses

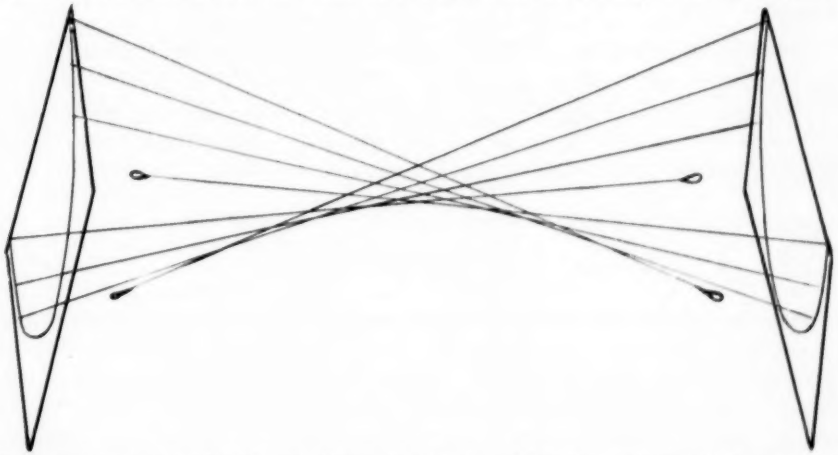

Fig. 8—Same hyperboloid as shown in Fig. 1. Only those lines which represent prestressing cables are shown

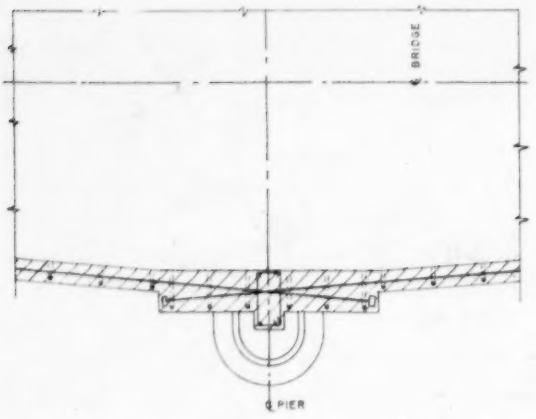

Fig. 9—Horizontal section at interior support, just below the deck slab, looking down

are maximum. These cables are to be placed in metal sheath in the forms, previous to the placement of concrete.

Placement of concrete

The curved part of the box has to be cast in place. After this has hardened, precast deck slabs (Fig. 10) may be tied down to the hyppar shape to form the top of the box. When all deck slabs are placed, prestressing reinforcement in the deck over the interior supports can be provided. After finishing the deck with cast-in-place concrete, leaving some room for stressing operations of cables in the deck, all main reinforcement can be stressed and falsework can be removed.

Design

Design procedures for a hyppar bridge are in principle the same as for any continuous prestressed concrete girder. Torsion, due to eccentric loading by trucks, must be considered. The structure has, however, a great torsional resistance due to its box shape. Transversely, the box is narrowest at mid-length; here the deck would have a maximum overhang.

CONCLUSION

Achievements in the last 10 years of long-span prestressed concrete bridge construction are truly remarkable, but there is no reason to assume that the development has come to an end. The several bridges which have been constructed up to now should be considered as pioneer structures. Yet to be exploited are the hyperboloid and paraboloid shapes, which have a potentially big role in future development of prestressed concrete structures and bridges in particular.

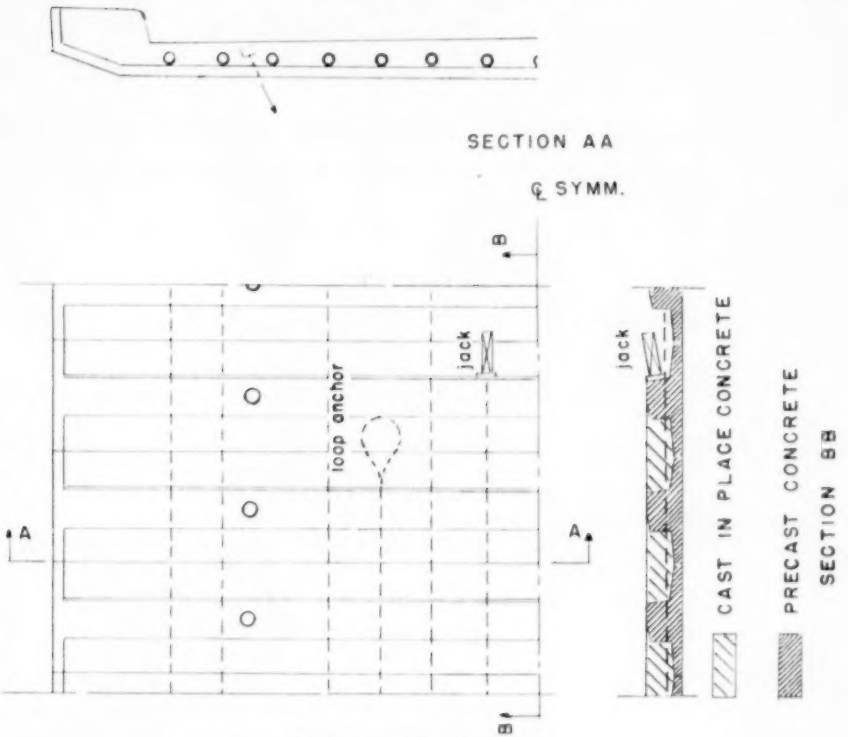

Fig. 10—Deck slab details

REFERENCES

1. Chaudisaigues, J., "La reconstruction en béton précontraint des ponts sur la Marne," *Annales de l'Institut Technique du Batiment et des Travaux Publics*, Jan. 1952.

2. Komendant, August E., *Prestressed Concrete Structures*, McGraw-Hill Book Company, Inc., New York, First Edition, 1952, Chapter 4, §43.

3. Lossier, H., Bonnet, M., and Thomas, L., "Le Pont de Villeneuve-Saint-Georges," *Annales de l'Institut Technique du Batiment et des Travaux Publics*, July-Aug. 1953.

4. "Cable Trusses Prestress Bridge," *Engineering News-Record*, Apr. 30, 1953, p. 50.

5. Finsterwalder, U., "Free-Span Prestressed Concrete Bridge," ACI JOURNAL, Nov. 1952, *Proc.* V. 49, pp. 225-232.

6. Leonhardt, Fritz, "Continuous Prestressed Concrete Beams," ACI JOURNAL, Mar. 1953, *Proc.* V. 49, pp. 617-634.

7. Saenz, Luis, and Martin, Ignacio, "Prestressed Concrete Bridge in Cuba Spans Nearly 300 Feet," *Civil Engineering*, V. 25, No. 12, Dec. 1955, p. 846.

8. Laponche, Raymond, and Arnoult, Jacques, "Reservoir hyperbolique a paroi mince en béton précontraint," *Travaux*, No. 161, Mar. 1948, p. 210.

9. Torroja, E., "Réalisations de voutes minces en Espagne, Résultats obtenus et possibilités futures," *Annales de l'Institut Technique du Batiment et des Travaux Publics*, New Series No. 164, Dec. 1950, pp. 19-23.

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Title No. 53-61

Diagonal Tension Strength of Reinforced Concrete T-Beams with Varying Shear Span*

By A. F. AL-ALUSI†

SYNOPSIS

A study of the influence of variations of shear span and percentage of longitudinal steel on mode of failure, cracking strength, and ultimate strength of reinforced concrete T-beams without web reinforcement is reported. Emphasis was placed on beams with large shear spans. The 25 beams tested included only simple-span beams under two symmetrical concentrated loads. The influence of mesh reinforcement in the flange, and that of longitudinal compression reinforcement was also investigated.

For a shear span ratio (a/d) between 4.0 and 8.0, the ratios v_u/f_c', ultimate shearing stress to cylinder strength, and v_{cr}/f_c', cracking shearing stress to cylinder strength, were for all practical purposes constant. For the same a/d range, a large increase in the steel area did not affect either the ultimate shearing strength or the cracking strength.

INTRODUCTION

Reinforced concrete beams in which the ratio of shear span‡ to the effective depth of beam is large fail in moment (usually in tension). This type of failure has been subjected to comprehensive tests.

If, however, the shear capacity of a beam is smaller than its flexural capacity, that is, if it is loaded at a smaller a/d value, three modes of shear§ failure are believed to be possible other than by bond failure:

(1) For reinforced concrete beams with very low values of the ratio of shear span to effective depth, a/d, the beam will fail without developing the ordinary beam action. This mode of failure is called shear proper; that is, the actual shearing off of the concrete compression zone along the line which was a continuation of the diagonal crack that started at or near the tension surface.

(2) Reinforced concrete beams in which the ratio of shear span to effective depth of beam is small, but larger than case (1) above, fail in shear-compression. Beams in this category show a diagonal crack at a load much lower than their maximum shear capacity, but final failure occurs with the flexural destruction of the compression zone above the main diagonal crack.

(3) Reinforced concrete beams in which the ratio of shear span to effective depth of beam is in the transition zone between those failing in shear-compression and those failing in moment (tension) fail in ordinary diagonal tension.

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‡The shear span, a, is defined as the distance from the plane of the nearest concentrated load point to the plane of support.

§Laupa, A., Siess, C. P., and Newmark, N. M., "Strength in Shear of Reinforced Concrete Beams," *Bulletin* No. 428, University of Illinois Engineering Experiment Station.

Earlier tests concerning the diagonal tension strength of reinforced concrete T-beams which fall in this transition region are not numerous enough to be conclusive.

Object and scope of study

Primary objectives of the investigation were:

(1) To investigate the influence of variations of the a/d ratio on the mode of failure, cracking strength, and ultimate strength of simple-span reinforced concrete T-beams without web reinforcement, concentrating on a/d ratios in and near the transition region between shear-compression and moment tension failures.

(2) To investigate the influence of variations in longitudinal tension reinforcement on the mode of failure, ultimate strength, and cracking strength, of these beams.

Secondary objectives were:

(1) To determine the effect of mesh reinforcement in the flange on the behavior, cracking strength, and ultimate strength of these beams.

(2) To investigate the influence of compression reinforcement on the behavior, cracking strength, and ultimate strength of these beams.

(3) To investigate the behavior, cracking strength, and ultimate strength of inverted T-beams.

The tests reported were selected primarily to cover the transition zone between shear-compression and moment failures, but enough specimens were included to tie in with the adjacent shear-compression region, and the transition points from diagonal tension failure to moment tension failure.

TEST PROGRAM

Test specimens

The beam cross section (Fig. 1) and concrete strength were kept constant except for small unintentional variations.

Three different amounts of longitudinal tensile reinforcement were used. Two #3 bars were used in seven beams; two #4 bars were used in 15 beams; and two #5 bars were used in three beams. All bars were intermediate grade billet steel with a yield point of approximately 53,000 psi and met ASTM A 305-53T. Five of the beams were reinforced with compression steel equal to their tension steel; three of these beams were tested as inverted T-beams and the other two were tested as regular T-beams. Mesh reinforcement was used in the compression flanges of six beams as indicated in Table 1.

Twenty-two test beams were designed to fail in diagonal tension by: (1) providing sufficient longitudinal tension reinforcement to prevent failure in moment tension; (2) furnishing sufficient flange area to prevent failure in compression; (3) providing end anchorages, in the form of steel plates welded to the end of the reinforcing bars, to prevent bond failures; (4) omitting all stirrups.

The other three beams were designed to fail in moment tension according to the plastic theory by Whitney.

The following construction features were maintained for these tests:

Aggregate size (maximum): ¼ in.
Cement: High-early-strength
Concrete slump: ½ to 1 in.
Concrete strength nearly constant (f_c' = 3650 to 4600 psi)
Curing: 1 day in wood forms, 6 days in moist room, 1 day in air of laboratory; tested at 8 days

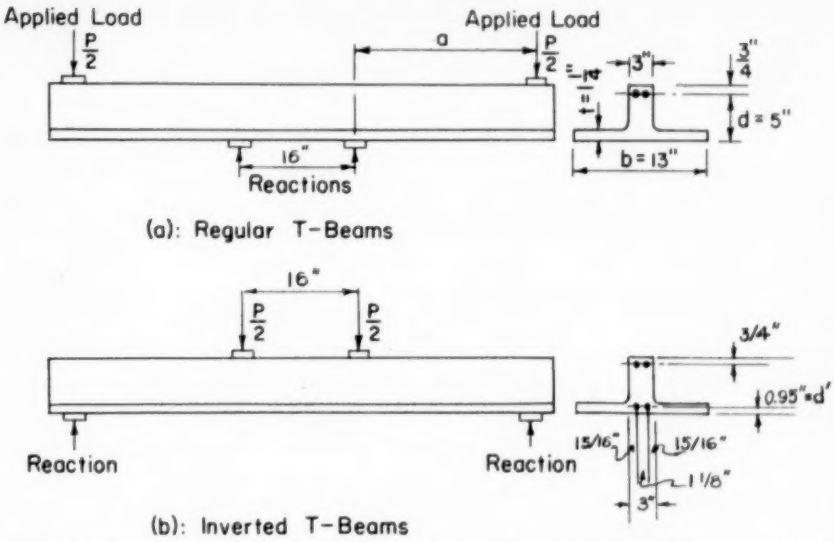


Fig. 1—Arrangement of beams during testing and cross section of beams. Loading blocks were 2½ in. wide, 8 in. long, ¾ in. thick.

Beam dimensions: $b = 13$ in.; $b' = 3$ in.; $d = 5$ in.; $t = 1\frac{1}{4}$ in.; over-all beam length varied between 42 and 92 in.

Concrete was vibrated in place.

Mesh reinforcement of 3 in. x 3 in. x 0.136-in. diameter was used in the compression flanges of six beams.

With each test beam cast, there were two modulus of rupture beams and four cylinders. These were kept and tested under the same conditions as for the test beams.

TEST PROCEDURE

Beams were loaded with two equally concentrated loads, the load being applied in increments by a screw-type testing machine. The crack pattern was carefully mapped on the whitewashed beam surface. To ease the tracing of the cracks during testing, all beams except No. 1 and the inverted T-beams were turned upside down as shown in Fig. 1. This arrangement placed the reactions near the middle of the beam and the loads at the ends (for regular T-beams). Particular attention was directed to find, as closely as possible, the location of the first diagonal crack and the load under which this crack started. Midspan deflections were recorded. After the test, the actual dimensions of the cross section of each beam were measured and used in the analysis.

Standard compression cylinders and modulus of rupture beams were cured exactly like the T-beams, and all were tested on the same day.

DATA

Test results are summarized in Table 1. Although it was intended to keep the concrete cylinder strength f_c' constant, the actual values varied from 3650 to 4600 psi with an average of 4020 psi. The a/d ratio varied from 2.00 to 7.76 with a majority of the beams having an a/d ratio greater than 4.00.

TABLE 1—DATA FROM TESTED BEAMS

Beam No.	a, in.	$\frac{a}{d}$	$P_w = \frac{A_s'}{b'd'}$, percent	$P_w' = \frac{A_s'}{b'd'}$, percent	Mesh reinforcement	Cylinder f_c', psi	Modulus of rupture, psi	v_u,* psi	v_{cr},† psi	$\frac{v_u}{f_c'}$	$\frac{v_{cr}}{f_c'}$	Remarks‡
6§	10.0	2.00	1.48	0	yes	3935	445	621	228	0.158	0.158	
12§	12.5	2.45	2.63	0	no	3650	386††	462	255	0.126	0.070	
11§	17.0	3.32	2.63	0	no	4150	512	298	237	0.072	0.057	
2	17.0	3.36	1.47	0	yes	4043	457	247	203	0.061	0.050	
21**	17.0	3.45	2.74	2.74	no	4240	510	412	256	0.097	0.060	Inverted T-beam
15**	20.0	3.96	2.67	2.67	no	3660	470	295	226	0.080	0.062	Inverted T-beam
3	20.0	4.00	1.49	0	yes	3945	467	245	189	0.062	0.048	
10	20.0	4.00	2.70	0	no	4150	470	252	205	0.061	0.049	
4	20.0	4.01	1.50	0	no	3850	472	238	194	0.062	0.050	
13§	20.0	4.02	2.72	0	yes	4165	464	291	189	0.070	0.045	
5	21.0	4.18	1.48	0	no	3950	512‡‡	243	185	0.062	0.047	
18	22.5	4.39	2.65	0	yes	3900	485	240	177	0.062	0.045	
1	22.5	4.45	1.48	0	no	3925	445	216	185	0.055	0.047	Moment tension failure
22	22.5	4.50	1.49	0	no	4150	461	223	174	0.054	0.042	Moment tension failure
7	22.5	4.50	2.71	0	no	3690	443	232	178	0.063	0.048	
24	22.5	4.51	2.71	2.71	no	4130	500	264	186	0.064	0.045	
16**	22.5	4.54	2.72	2.72	no	4080	478	254	192	0.062	0.047	Inverted T-beam
17	27.5	5.36	2.65	0	yes	4290	511	237	197	0.054	0.045	
8	27.5	5.50	2.71	0	no	3810	465	230	187	0.060	0.049	
19	27.5	5.50	4.21	0	no	4440	506	240	186	0.054	0.042	
25	27.5	5.79	2.85	2.85	no	3770	442	232	178	0.062	0.047	
9	32.5	6.50	2.71	0	no	4600	490	242	198	0.053	0.043	
20	32.5	6.54	4.21	0	no	3955	503	257	176	0.065	0.045	
14	37.0	7.44	2.70	0	no	3920	492	217	189	0.055	0.048	Moment tension failure
23	37.0	7.76	4.40	0	no	4090	472	240	183	0.059	0.045	
							Avg 476					

*$v_u = \frac{8}{7}\frac{V_u}{b'd}$

†$v_{cr} = \frac{8}{7}\frac{V_{cr}}{b'd}$

‡All test beams failed in diagonal tension except for the three noted as moment tension failures.
§Designated as Group II in discussion.
**Designated as Group III in discussion.
Those without § or ** constitute Group I (moment tension failure excluded).
††Modulus of rupture beams were left in open air about 48 hr longer than other modulus of rupture beams.
‡‡Modulus of rupture beams were left in open air about 10 hr less than other modulus of rupture beams.

The ultimate shearing stress v_u was calculated from the relationship: $v_u = 8/7\ (V_u/b'd)$, where V_u is the maximum shear force that the beam sustained. The cracking shearing stress v_{cr} was calculated from the same relationship using the cracking shear V_{cr}.

Failure behavior of test beams

The behavior of all beams was essentially elastic until tension cracks formed. Moment cracks started near the midspan on the tension surface of the beams and extended vertically toward the compression surface. As the load increased, new moment cracks developed between midspan and the reaction, and the height of the early moment cracks increased. A further increase in load caused the moment cracks between load points and reactions to curve toward the center reaction (Fig. 2). As the load increased, separate 45-deg diagonal cracks formed between the longitudinal tension reinforcement and middepth of the beam. The first one of these separate diagonal cracks is designated as the initial diagonal crack, and the corresponding stress is called the cracking shearing stress v_{cr}. Under further increase in load, new diagonal cracks (about 30 deg or less with beam axis) formed. It was noted that these flat diagonal cracks were the cracks which caused final failure.

Test beams of this investigation which failed in diagonal tension can be divided into three groups according to their behavior just before failure. In 15 beams, which make up Group I, an initial diagonal crack (inclined at about 45 deg) was first formed at each end of the beam followed by a flatter diagonal crack (inclined at about 30 deg or less with beam axis) at one end only. The flat diagonal crack was formed suddenly and its formation was simultaneous with failure. Beam 17, Fig. 3, illustrates the above type of failure and is considered typical for beams of Group I. It is noted in this figure that the diagonal tension crack was a continuous crack all the way to the compression surface. There was no evidence of a local compression failure.

In each beam of Group II, which included Beams 6, 11, 12, and 13, an initial 45-deg diagonal crack was formed first at each end of the beam, symmetrically, and under increased load a flatter diagonal crack (30 deg or less with beam axis) was formed at each end (Fig. 2). These flat diagonal cracks were formed progressively, not suddenly, and their formation was not simultaneous with failure. Group II beams showed no evidence of local compression

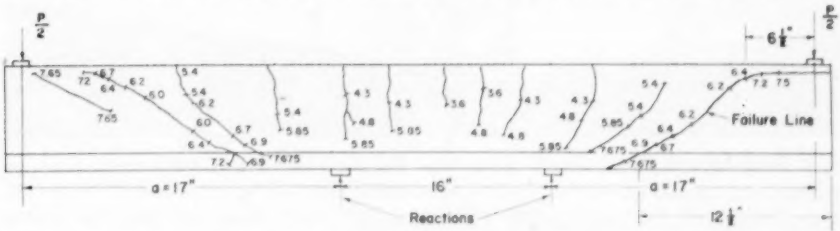


Fig. 2—Cracks in Beam 11, which is typical for beams in Group II. Details: $a/d = 3.32$; $P_w = 2.63$ percent; no mesh reinforcement in flange; tested as regular T-beam; diagonal tension failure

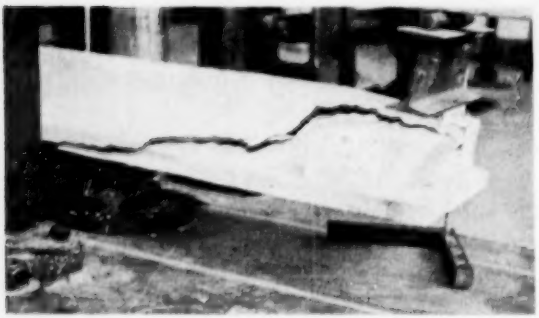
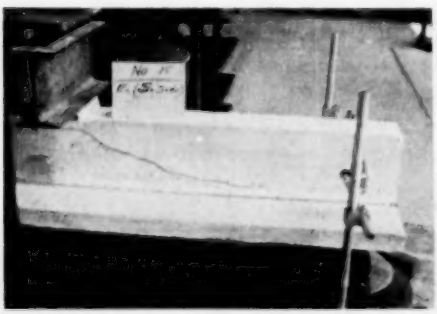


Fig. 3—Beam 17, typical for beams of Group I. Note how the diagonal tension crack was a continuous crack all the way to the compression surface

failure. Due to the presence of these two 30-deg symmetrical cracks, i.e., one at each end, beams of this group in general had higher ultimate shearing stress than those of Group I, as is shown in Table 1. All beams of Group II had a/d ratios of 4.0 or less.

Group III included three inverted T-beams, No. 15, 16, and 21. In Beam 15, when the load reached 5.7 kips, a 45-deg crack started at the east end. When the load increased to 6.0 kips, a similar diagonal crack appeared at the west end. These two initial diagonal cracks were much closer to the points of applied load than to the reactions (Fig. 4 and 5). Under further increase of load, each of the initial cracks passed under the bearing plate of the applied load in one end and branched into a 30-deg crack on the other end. When the load reached 6.37 kips, a flat diagonal crack (inclined less than 30 deg with beam axis) formed at the west end and the beam carrying capacity dropped to 5.7 kips. This first looked like a failure, but a favorable internal redistribution of stresses* caused the beam to sustain a maximum load of 7.5 kips. The general behavior of Beam 21 was similar to that of Beam 15. Both these beams seemed to act as tied arches after the formation of two symmetrical



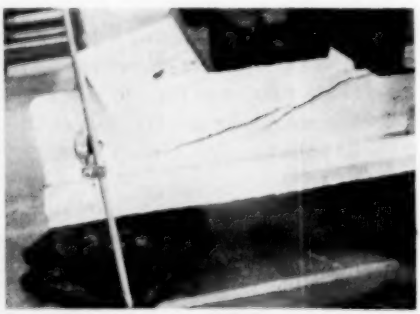


Fig. 4 (left)—Beam 15 (inverted) shows how the diagonal cracks were much closer to the points of applied load than to the reactions. Fig. 5 (right)—Beam 15 (inverted) shows that under high loads the initial diagonal crack branched into flat cracks

*The phenomenon of redistribution of internal stresses caused by diagonal tension cracks was presented for the first time by E. Mörsch and was discussed by Moody, K. G., Viest, I. M., Elstner, R. C., and Hognestad, E., "Shear Strength of Reinforced Concrete Beams. Part 1—Tests of Simple Beams," ACI JOURNAL, Dec. 1954, *Proc.* V. 51, p. 329.

flat diagonal cracks, i.e., one at each end. Although Beam 16 was an inverted T-beam and the progress of diagonal cracks was in general similar to that of Beams 15 and 21, it did not seem to act as a tied arch.

Type of failures

Twenty-two T-beams failed in diagonal tension. In each one of these beams more than one diagonal crack was formed before the ultimate capacity was reached. The other three beams, No. 1, 22, and 14, failed in moment tension.

Beam 5 having an a/d ratio of 4.18 seemed to lie at the boundary between diagonal tension and moment tension failures for its percentage of longitudinal tension reinforcement, P_w*, of 1.48 percent. In this beam the longitudinal tension steel yielded before the final diagonal tension failure occurred.

Observation of all beams which failed in diagonal tension showed that failure occurred when a diagonal crack formed and continued all the way to the compression surface, as shown in Fig. 3. There was no apparent local compression failure. This observation was true for regular T-beams and for inverted T-beams.

Beam 9 having an a/d ratio of 6.5, and $P_w = 2.71$ percent seemed to lie at the boundary between diagonal tension failure and moment failure for this steel percentage.

DISCUSSION OF RESULTS

a/d ratio

Emphasis was placed on beams with large shear span ratios (that is, a/d ratio between 4.0 and 8.0). Fig. 6 shows the ratio V_{cr}/f_c' versus the a/d ratio. Fig. 7 shows two curves for the v/f_c' values at first diagonal crack and at failure.

When the a/d ratio lies between 4.0 and 8.0, Fig. 6 and 7 show that the v/f_c' values at first diagonal crack and at failure are essentially constant. All values fall within ± 10 percent of the following:

$$\frac{v_{cr}}{f_c'} = 0.046 \qquad (1)$$

$$\frac{v_u}{f_c'} = 0.0625 - 0.0002\,(a/d) \qquad (2)$$

Eq. (2) was derived by the method of least squares. Since the last term on the right hand side of Eq. (2) is extremely small, it can be neglected for practical purposes.

It is noted, from Fig. 7, that when the a/d ratio was between 2.0 and 4.0, the ratio v_u/v_{cr} decreased with increasing a/d. When the a/d ratio was between 4.0 and 8.0, the ratio v_u/v_{cr} is essentially constant.

*$P_w = \frac{A_s}{b'd}$, $P_w' = \frac{A_s'}{b'd}$.

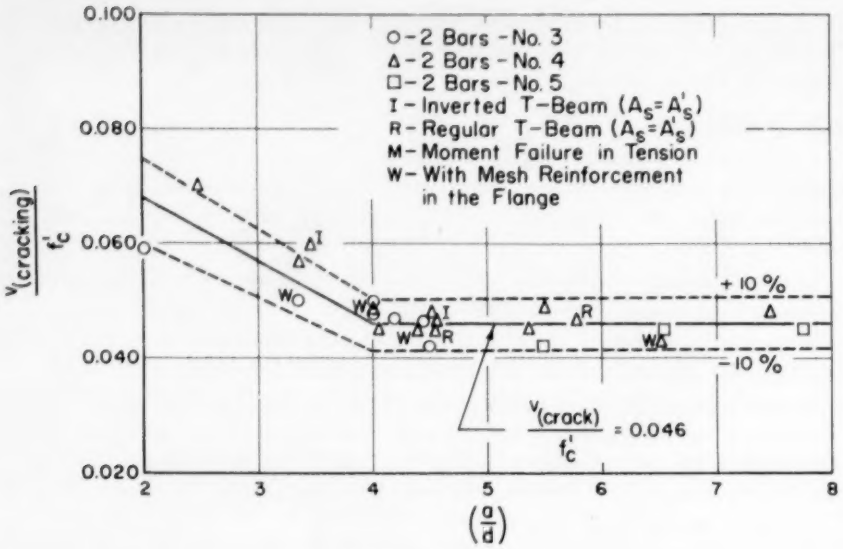


Fig. 6—v_{cr}/f_c' versus a/d*

*Equations for $a/d = 2$ to 4 were derived by the method of least squares. However, since the tests in this range were limited, the equations are not presented in this paper.

Longitudinal tension reinforcement

It was originally intended to study the influence of different percentages of longitudinal tension reinforcement on the ultimate shearing strength of the tested beams. This percentage, P_w, varied from 1.47 to 4.40 as shown in Table 1. Fig. 6 and 7 show that when the a/d ratio was between 4.0 and 8.0 and when other factors remain the same, variation in the percentage of longitudinal tension reinforcement does not affect either the ultimate or the cracking shearing stresses significantly.

Fig. 8 shows that the type of failure, i.e., whether it is diagonal tension or moment tension, depends upon both the a/d ratio and the percentage of longitudinal tension reinforcement P_w. In this figure, two curves were drawn according to Whitney's theory based on P_w values of 1.49 percent (two #3 bars) and 2.71 percent (two #5 bars). Six beams were tested having $P_w = 1.49$ percent. Beams 1 and 22 failed in moment tension; Beam 5 was yielding before the final failure in shear occurred. Beams 2, 3, and 4 did not yield; however, the moment tension cracks extended along the full depth of the stem of the beam but never penetrated into the flange. The test results reported here fall close to and slightly above those predicted by Whitney.

Longitudinal compression reinforcement

Five test beams were equally reinforced in tension and compression. Three of these, No. 15, 16, and 21, were tested as inverted T-beams; the other two, No. 24 and 25, were tested as regular T-beams.

The test results of Beams 16, 24, and 25 (with a/d ratios greater than 4.0), as given in Table 1, did not show any increase in ultimate shearing stress due to the presence of the compression reinforcement.

Two of the inverted T-beams, No. 15 and 21 (in which a/d ratios were less than 4.0), behaved in a distinctly different manner from all the other beams tested. A comparison between the ultimate shearing stresses v_u for these two inverted T-beams, and that of regular T-beams (with approximately equal a/d ratios), shows that the v_u for the two inverted T-beams was higher than that for other T-beams. The reason for this was not established.

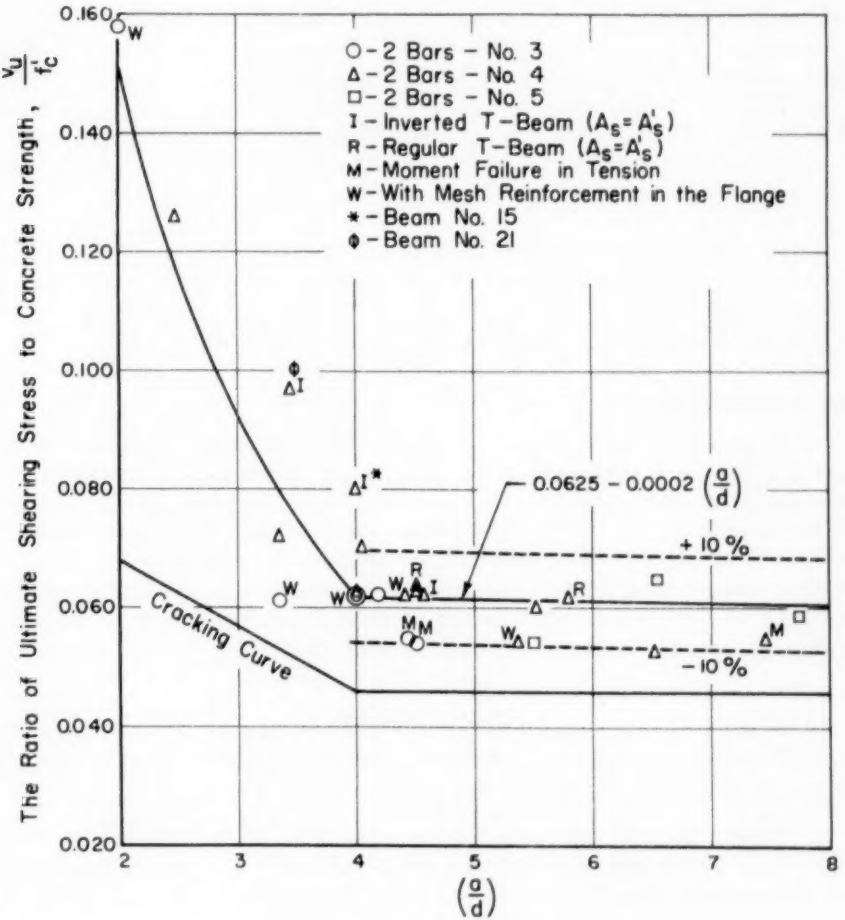


Fig. 7—v_u/f_c' versus a/d*

*Equations for a/d = 2 to 4 were derived by the method of least squares. However, since the tests in this range were limited, the equations are not presented in this paper.

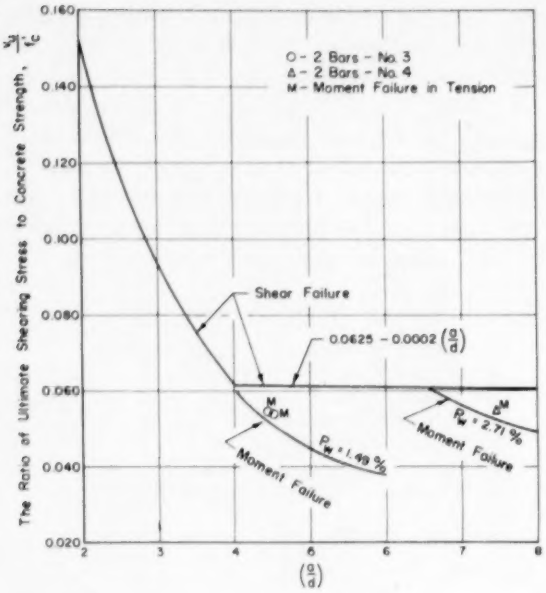


Fig. 8—v_u/f_c' versus a/d* and curves for theoretical moment failure in tension

*Equations for a/d = 2 to 4 were derived by the method of least squares. However, since the tests in this range were limited, the equations are not presented in this paper.

Mesh reinforcement

Mesh reinforcement was used in the compression flanges of six beams as given in Table 1. Test Beam 13, with mesh reinforcement, showed some increase over its companion without mesh reinforcement. However, five other test beams, marked W in Fig. 6 and 7, provided with mesh reinforcement, did not show any increase in the ultimate shearing strength over their similar beams without mesh reinforcement. For this reason it was concluded that the presence of such mesh reinforcement in the compression flange of T-beams did not affect the ultimate strength in shear.

CONCLUSIONS

The primary conclusions derived from this investigation, for beams which failed in diagonal tension, within the limits of the tests, were:

1. When the a/d ratio was between 4.0 and 8.0, the ratios v_{cr}/f_c' and v_u/f_c' were for all practical purposes constant as shown in Fig. 6 and 7 and as expressed by Eq. (1) and (2) respectively.

2. When the a/d ratio was between 4.0 and 8.0, the variation in the percentage P_w of longitudinal tension reinforcement (to web area) from 1.47 to 4.40 percent did not affect either the ultimate shearing strength or the cracking strength.

In this transition zone between shear-compression failure and moment tension failure, the criterion of failure in diagonal tension appears to be the attainment of a specific critical shearing unit stress.

The secondary conclusions were:

1. The presence or absence of mesh reinforcement in the compression flange of T-beams affected neither the ultimate shearing strength nor the cracking strength.
2. The presence of compression reinforcement affected neither the ultimate shearing strength nor the cracking strength.
3. Observation of all beams which failed in diagonal tension showed that failure occurred with a diagonal crack continuing all the way to the compression surface. There was no evidence of a local compression failure. This observation was true for regular T-beams and for inverted T-beams.
4. When the a/d ratio was between 4.0 and 8.0, the actual ultimate moment increased in direct proportion with the a/d ratio. This can be deduced from the fact that in this a/d range, the ultimate shearing stress was practically constant.
5. (a) When the a/d ratio lies between 4.0 and 8.0, the ratio of v_u/v_{cr} is for all practical purposes constant at 1.35.

   (b) When the a/d ratio lies between 2.0 and 4.0, the ratio v_u/v_{cr} decreases with an increase in the a/d ratio.
6. The inverted T-beams behaved somewhat differently from the regular T-beams. Since the number of specimens tested as inverted T-beams was small, additional tests are needed.

ACKNOWLEDGEMENT

This paper is based upon a dissertation by the author, submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy in Civil Engineering in the Graduate School of the University of Texas, 1955. The investigations for this dissertation were carried out under the guidance of Prof. Phil M. Ferguson.

Discussion of this paper should reach ACI headquarters in triplicate by Sept. 1, 1957, for publication in the Part 2, December 1957 JOURNAL.

CONCRETE BRIEFS

Notes from Field and Office

Recommendations of the International Subcommittee on Concrete for Large Dams on the Definition of Concrete Mixes Used in Dam Construction.....1079
Column Design for Combined Bending and Axial Load—Uncracked Sections 1081
Problems and Practices.....1083

Recommendations of the International Subcommittee on Concrete for Large Dams on the Definition of Concrete Mixes Used in Dam Construction*

The purpose of this report is to establish a uniform international procedure for reporting the composition of concrete mixes used in the construction of dams and other important structures. Such a uniform procedure would greatly assist in the studies of the performance of dams and structures on a world-wide basis.—EDITOR

* * *

In comparing a concrete described in one published report with that described in another, difficulties are often experienced owing to the absence in either or both reports of certain of the relevant data. This is especially so in comparing concretes for dams, partly due to the complexity introduced by their special characteristics, *e.g.*, the use of aggregates of large maximum size, the use of lean mixes, and partly due to the number of aspects from which comparisons may eventually be required, *e.g.*, strength, permeability, heat evolution, frost resistance. To assist those compiling reports on concretes for dams in deciding what data to include, and to encourage the presentation of data likely to be of use in future researches, the International Subcommittee on Concrete for Large Dams has prepared lists of the minimum data which it recommends should be included in future published reports. No finality can, of course, be claimed for such lists; their extension must depend on future developments in concrete technology.

While it is obviously desirable that a full report should be published on the concrete used in each dam, it is recognized that on occasions only a brief description of the concrete can be included. It is recommended that each type of report should include the following minimum data.

1. Minimum data required in a brief description of the concrete in a dam

(a) Cement content of finished concrete (in kg per cu m or lb per cu yd).

(b) Effective water-cement ratio (*i.e.*, corrected for aggregate in the saturated, surface-dry condition).

(c) Type or types of cement with reference to any standard specifications applicable. (In the case of "blended" cements, the proportion by weight of pozzolan or other material present should be stated.)

(d) Broad petrological classification of aggregate, *e.g.*, limestone, granite, etc., origin (*i.e.*, crushed or natural), and maximum size employed.

(e) Method of compaction.

(f) Use of a workability aid or air-entraining agent.

(g) Estimated air content of finished con-

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*Paper No. 11, *Report* R70, presented by International Subcommittee on Concrete for Large Dams at 1955 Congress, Paris, of the International Commission on Large Dams.

crete, whether entrained or entrapped, as a percentage by volume.

2. Data required in a full report on the concrete in a dam

(*1*) *Cement*

(a) Type or types of cement with reference to any standard specifications applicable. (In the case of "blended" cements, the proportion by weight of pozzolan or other material present should be stated.)

(b) Specific gravity of cement, *e.g.*, as determined in distilled kerosene.

(c) Chemical composition of cement, including percentage of alkalies present.

(d) Properties of cement, including strength, fineness, and heat of hydration. (The method of test should be briefly indicated with reference to any standard specification applicable.)

(*2*) *Admixtures* (including workability aids, air-entraining agents)

(a) Nature of added material.

(b) Amount of added material (usually in terms of a unit weight of cement).

(*3*) *Properties of aggregate materials* (*including sand or inert additions*)

The following data are required on the aggregate materials; where a concrete mix contains material from more than one source (*e.g.*, where a natural sand from a pit or river deposit is used in conjunction with crushed rock from a quarry) the data should be given separately for each source of material, irrespective of any overlap in particle sizes.

(a) Petrological type and description, origin (*i.e.*, crushed or natural), particle shape, and surface texture.

(b) Size distribution and proportion by weight of the total aggregate in the mix.

(c) Absorption of water by weight from the oven-dry (100 to 110 C) condition to the saturated, surface-dry condition.

(d) Coefficient of thermal expansion of aggregate material (more especially for concretes exposed to frost action).

(*4*) *Combined aggregate* (as used in the concrete mix)

(a) Maximum size of aggregate used.

(b) Complete grading analysis of the combined aggregate expressed in terms of "dry" weights [see 4(c) below]. (The grading analysis should cover the whole range of aggregate sizes down to the finest sand; it should include a statement of the percentage by "dry" weight of fine material passing the ASTM No. 4, or equivalent, sieve.)

(c) Absorption of water by weight, from the "dry" condition (*i.e.*, as used in the grading analysis) to the saturated, surface-dry condition, of the fractions above and below the ASTM No. 4, or equivalent, sieve.

(The "dry" condition customarily used in the grading analysis may differ in different countries and may range from the oven-dry to the saturated, surface-dry condition. The data on absorption are necessary in order that the mix may eventually be expressible in terms of the absolute volumes occupied in the finished concrete.)

(d) Specific gravities in the saturated, surface-dry condition of the fractions above and below the ASTM No. 4, or equivalent, sieve.

(e) Method of batching (*i.e.*, by weight or volume); size limits of the aggregate fractions which are batched separately.

(*5*) *Concrete proportions*

In the concrete mix:

(a) Aggregate-cement ratio by weight, stating whether the ratio is based on aggregate in the "dry" condition (see Section 4 above) or has already been corrected for the saturated, surface-dry condition using the data already given.

(The proportion of aggregate passing the ASTM No. 4, or equivalent, sieve will already have been given in Section 4(b) above.)

(b) Effective water-cement ratio by weight (*i.e.*, corrected for aggregate in the saturated, surface-dry condition).

In the finished concrete (*i.e.*, in the concrete in place in the dam after compaction but before any water loss through drying):

(c) Estimated cement content of the finished concrete (in kg per cu m or lb per cu yd).

(d) Estimated air content of finished concrete, whether entrained or entrapped, as a percentage by volume.

(e) Estimated density of finished concrete.

(These three estimated quantities are interrelated and will, in many cases, be based on the measured volume of the finished concrete as compared with the amount of cement used. They should be consistent both with each other and with the data already given on the mix proportions by weight and by absolute volume.)

(6) *Workability and method of compaction*

(7) *Properties of concrete and quality control*

(a) Dimensions, preparation, and curing of control cubes, prisms, or cylinders.

(b) Statistical variation in strength or of other properties measured. (This should include the number of specimens tested, the average strength or other property measured, and the coefficient of variation.)

(c) Method of removing the large aggregate from the concrete used for test specimens when aggregate above a certain size is excluded.

(d) Measured air content of mix, *e.g.*, as measured on a sample of fresh concrete in an air meter.

(e) Measured density of mix, *e.g.*, as measured on a concrete specimen.

General note on list 2

Care should be taken that sufficient of these data are included to permit calculation of the composition, in terms of the absolute volumes occupied in the finished concrete by the cement, aggregates, water and air, of both: (a) the total concrete, and (b) that part of the concrete material below the ASTM No. 4, or equivalent, sieve size (*i.e.*, excluding the larger aggregate).

(Inclusion of the data under heading (b) provides for the possibility of comparing different concretes containing large size aggregates through the properties of their matrix mortars.)

Where the composition of the concrete by absolute volume has been calculated, it should be given in addition to the more conventional mix data.

Column Design for Combined Bending and Axial Load—Uncracked Sections

By MANUEL SAVRAN*

The ACI Code column formulas

$$f_c = \frac{N}{A_g}\left[\frac{1 + De/t}{1 + (n-1)p_g}\right] \quad \text{Section 1109, Eq. (27)}$$

$$f_p = f_a\left[\frac{1 + De/t}{1 + CDe/t}\right] \quad \text{Section 1110, Eq. (29)}$$

} ACI 318-51

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leqq 1 \quad \text{Section 1109, Eq. (18) ACI 318-56}$$

for combined bending and axial load, uncracked sections, when transformed into the following convenient form

$$\frac{N}{A_g} = \frac{0.225f_c' + f_s p_g}{1 + CDe/t} \times 0.8 \text{ (tied columns)}\dagger$$

can be easily expressed graphically. Values of C and D refer to notation in ACI 318-51.

Values of N/A_g are plotted against De/t for various steel percentages in Fig. 1 for $f_c' = 3000$ and $f_s = 16{,}000$. Graphs for any other combination of f_s and f_c' can be similarly made.

The simplicity of design is best shown by the following numerical example.

Example

Design a tied column for

$M = 150$ ft-kips $\quad f_s = 16{,}000$ psi

$N = 300$ kips $\quad f_c' = 3000$ psi

$n = 10$

$$e = \frac{150 \times 12}{300} = 6 \text{ in.}$$

when $De/t = 0$. Assume $p = 0.02$. From Fig. 1, $N/A_g = 0.8$ approximately.

$$A_g = \frac{300}{0.8} = 375.$$

Try 20 x 20 in. column: assume $D = 5$.

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†See appendix for derivation.

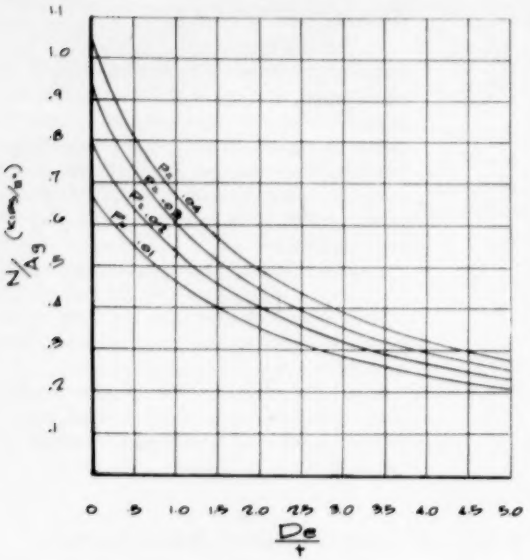

Fig. 1—Values of p for tied column—$f_s = 16{,}000$, $f_c' = 3000$, $n = 10$

$$De/t = \frac{5 \times 6}{20} = 1.5$$

$$N/A_g = \frac{300}{400} = 0.75$$

From Fig. 1, $p > 0.04$.

Try 22 x 24 in. column: assume $p = 0.02$, $D = 4.9$.

$$De/t = \frac{4.9 \times 6}{24} = 1.22$$

$$N/A_g = \frac{300}{528} = 0.57$$

From Fig. 1, $p = 0.031$. A_s = ten #11 bars (five in each face). Actual $p = 0.0295$.

Checking the above results with Eq. (27) and Eq. (29) in ACI 318-51:

$$D = 4.6, \quad f_a = 727, \quad C = 0.54$$

$$f_c = \frac{300{,}000}{528}\left[\frac{1 + 4.6 \times 6/24}{1 + (10 - 1)0.0295}\right]$$

$$= 966 \text{ lb per sq in.}$$

$$f_b = 727\left[\frac{1 + 4.6 \times 6/24}{1 + 0.54 \times 4.6 \times 6/24}\right]$$

$$= 963 \text{ lb per sq in.}$$

Checking with Eq. (18) in ACI 318-56:

$$I = 1/12 \times 22 \times 24^3 + 9 \times 15.6 \times (9\tfrac{1}{2})^2 = 38{,}000 \text{ in.}^4$$

$$f_a = \frac{300{,}000}{528} = 568 \text{ lb per sq in.}$$

$$F_a = (0.225f_c' \times f_s p)0.8 = 920 \text{ lb per sq in.}$$

$$f_b = \frac{Mc}{I} = \frac{150{,}000 \times 12 \times 12}{38{,}000}$$

$$= 568 \text{ lb per sq in.}$$

$$F_b = 0.45f_c' = 1350 \text{ lb per sq in.}$$

$$\frac{568}{920} + \frac{568}{1350} = 1.039$$

It should be noted that although the value C appears in the equation, allowance has been made for it in the graph and there is therefore no need for computing it.

The advantage of using the procedure shown in this article becomes obvious when designing sections with steel on four faces. Values of D can be found by using the following formula:*

$$D = \frac{1 + (n - 1)p}{0.167 + (x + yz^2)(n - 1)pg^2}$$

The quantity $x + yz^2$ is taken from Table 1 of the paper by Cervin.*

*Cervin, D. R., "Design of Rectangular Tied Columns Subject to Bending with Steel in All Faces," ACI JOURNAL, Jan. 1948, *Proc.* V. 44, pp. 401-412.

Appendix

Derivation of formula

$$\frac{N}{A_g} = \frac{0.225f_c' + f_s p_g}{1 + CDe/t}$$

From ACI 318-51, Sections 1109 and 1110

$$f_c = \frac{N}{A_g}\left[\frac{1 + De/t}{1 + (n-1)p_g}\right] \quad \ldots\ldots (27)$$

$$f_a = \frac{0.225f_c' + f_s p_g}{1 + (n-1)p_g}$$

$$f_p = f_a\left[\frac{1 + De/t}{1 + CDe/t}\right] \quad \ldots\ldots (29)$$

$$f_c = f_p$$

$$\frac{N}{A_g}\left[\frac{1 + De/t}{1 + (n-1)p_g}\right]' =$$

$$\left[\frac{0.225f_c' + f_s p_g}{1 + (n-1)p_g}\right]\left[\frac{1 + De/t}{1 + CDe/t}\right]$$

$$\frac{N}{A_g} = \frac{0.225f_c' + f_s p_g}{1 + CDe/t}$$

Derivation from ACI 318-56, Section 1109

$$f_a = \frac{N}{A_g}$$

$$f_b = \frac{M\,c}{I} = \frac{NDe}{t\,A_g[1 + (n-1)p_g]}$$

$$F_a = 0.225f_c' + f_s p_g$$

$$F_b = 0.45f_c' = \frac{0.225f_c' + f_s p_g}{1 + (n-1)p_g} \times \frac{1}{C}$$

$$\frac{f_a}{F_b} + \frac{f_b}{F_b} \leqq 1$$

$$\frac{\dfrac{N}{A_g}}{0.225f_c' + f_s p_g} + \frac{\dfrac{NDe}{t\,A_g[1 + (n-1)p_g]}}{\dfrac{0.225f_c' + f_s p_g}{1 + (n-1)p_g} \times \dfrac{1}{C}} = 1$$

$$\frac{N}{A_g}\left[\frac{1}{0.225f_c' + f_s p_g} + \frac{CDe}{t(0.225f_c' + f_s p_g}\right] = 1$$

$$\frac{N}{A_g(0.225f_c' + f_s p_g}\left[1 + \frac{CDe}{t}\right] = 1$$

$$\frac{N}{A_g} = \frac{0.225f_c' + f_s p_g}{1 + CDe/t}$$

Problems and Practices

A series relating to "down-to-earth, everyday" concrete problems which attempts to give brief answers to the more common (and sometimes uncommon, too) questions asked about concrete design and construction practices.

To some, the answers will seem simple and obvious; others may prove to be extremely controversial.

All ACI members are invited to participate—either by submitting an inquiry, or even better, by telling JOURNAL *readers how an intriguing problem was solved. It may well be that readers will be able to suggest more practical solutions than those presented.*

Q. There has been considerable controversy on concrete test cylinders on this project, due largely to their treatment after casting. The owner's laboratory uses job-cured cylinders to represent concrete placed and not as a check on the design mix. I believe that any loss of strength from the design mix should be taken into account in design of the structure, which is normal practice. Therefore, cylinders should be given the curing conditions of the laboratory design cylinders and should produce the design strengths.

A. The design strength, f_c', may be defined as the assumed compressive strength of concrete after 28 days of standard (laboratory) curing, as measured by the standard test on standard (6 x 12-in.) cylinders. Specifications usually require that the average actual strength be somewhat higher to insure that no large amount of concrete consecutively placed be below the assumed strength for design (see Section 304, ACI 318-56). Differences in strength between the test cylinders and concrete in place are expected proportionate to the differences in curing conditions. Instead of regarding the usual safety factor as providing for a loss in strength due to job

curing conditions, most designers endeavor to specify a formal curing procedure which, combined with anticipated later intermittent curing due to exposure, will eventually develop strength of concrete in place equal to or greater than that specified.

To determine adequacy of concrete as furnished, only the standard laboratory curing will provide a basis for comparison of results. Note the statement in ACI 318-56, Section 304(c): "The strengths of cylinders cured on the job are intended to indicate the adequacy of protection and curing of the concrete, and may be used to determine when forms may be stripped, shoring removed, or the structure placed in service." If laboratory-cured cylinders develop less than specified strengths, the proportions of the concrete should be changed to increase the strength. If job-cured cylinders fail to develop the expected or specified strength, the contractor may be required to extend or improve his protection and curing procedures.

It would be quite proper for any architect or engineer in preparing specifications to require that job-cured specimens develop any fixed percentage of the 28-day strength of laboratory-cured cylinders before formal curing measures are terminated. Few specifications are complete or specific to this extent. There is no generally accepted practice to establish such a percentage. Unfortunately, the interpretation of tests on job-cured specimens is usually left open to argument and consequently practice varies.

* * * *

Q. The roof of a multi-story reinforced concrete flat-slab industrial building showed signs of imminent failure. These signs included noticeable sagging and cracking in the exposed soffit, leaks, and sagging noticeable from above, since every panel formed a "bird bath" holding rain water. The owner has been required to evacuate manufacturing processes from the floor beneath and to install timber shores for immediate safety. What salvage methods should be considered short of complete removal and replacement?

The physical data are: conventional flat slab construction with drop panel and mushroom capitals, slabs 7 in. thick, and 24-ft square bays. Interior downspouts are at alternate interior columns. Age is about 40 years. Top of slab shows 4 to 6 in. deep "bird baths" in every panel. Building code prescribes a snow load of 30 psf. Original construction and later repairs left following loads in order from bottom to top: 2 to 4 in. compacted cinder fill, 2 to 4 in. concrete, original built-up roofing, 2 to 10 in. compacted cinder fill, 2 in. concrete, and later built-up roofing, tar, and gravel. This total of approximately 100 psf dead load is now removed. No design or detail drawings available. Test holes chopped through at points of leakage show fair concrete quality, #5 deformed bars at 12 in. each way in bottom, (one rusted through, all others look perfect); circular crack pattern about 5 to 6-ft diameter at top of each column. Both top and bottom cracks visible, perhaps 1/16 in. wide.

A. Four solutions were considered:

1. Add a new bonded topping as thin as possible while roof is shored up completely; add new roof sumps at center of panels. This solution would remove perhaps 50 psf dead load in anticipation of the 30 psf live load capacity legally required. Proposed by owner; rejected, in worst part of roof at least, by the building officials, as not providing adequate safety, in view of possible and uncertain rust damage to reinforcement.

2. In addition to corrective measures proposed in (1), add permanent structural steel shoring consisting of beams under center of panels to posts resting on floor near concrete columns within area of capital under floor. Rejected on cost basis due to interference with overhead pipes, ducts, and conduit.

3. In addition to corrective measures proposed in (1), add concrete beams and girders on top of slab. Locate girders on each interior column center line in one direction, beams, on column center lines between new roof sumps and across centers of the worst panels. Fasten roof slab to new framing by threaded tie bolts through slab to bear on plate washers under slab, tightened from below after new beams have hardened.

4. Combination of corrective measures proposed in (1), (2), and (3). Substitute steel beams beneath slab for concrete beams proposed in (3) at center of panels. Hang steel beams under slab from concrete girders above slab. After tightening end hanger bolts for steel beams wedge for bearing at the center of panel. This would eliminate posts required in proposal (2) and provide better roof drainage than proposal (3).

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

New-type designs of unshored wooden centering for arched bridges (Neuartige Konstruktionen von freitragenden hoelzernen Lehrgeruesten fuer gewoelbte Bruecken)

K. Deininger, *Der Bauingenieur* (Berlin), V. 31, No. 2, Feb. 1956, pp. 69-73

Reviewed by Aron L. Mirsky

Theory, design, and construction of three types of timber centering, composed primarily of laminated timbers (bundles) held together by steel yokes. First and simplest type, for short spans, is the simple laminated arch; second uses a trussed arch, composed of two such arches as chords plus web members consisting of timber struts and held together by steel cables or rods; third type is the two-or-more story trussed arch.

Placing sequence for concrete arches using this centering is discussed.

See also *Engineering* (London), V. 180, No. 4688, Dec. 2, 1955, pp. 757-759 ("Current Reviews," ACI Journal, Apr. 1956, p. 893); and *Engineering News-Record*, V. 157, No. 7, Aug. 16, 1956, p. 71.

Prestressed concrete grid bridges with stiffened exterior beams

A. Gallia, *Constructional Review* (Sydney), V. 29, No. 10, Oct. 1956, pp. 23-32

Grid bridge is described as a simply supported highway bridge with two stiffened exterior beams and a large number of interior beams tied together with transverse cables through attached diaphragms only. No lateral post-stressing through the deck slab is necessary. The I-beams are precast, prestressed near the bridge site, placed side by side on abutments, and then hinged together with their upper flanges to form the bridge deck, but this deck slab does not contribute to the transverse stiffness of the bridge structure.

Purpose of this paper is to discuss principal variables influencing wheel load distribution and to recommend simple general design method based on Leonhardt's theory for a system in which the exterior beams possess greater stiffness than the interior beams. Practical examples explain proposed method.

Prestressed concrete bridge across the River Mahi at Vasad

M. V. Joglekar, *The Indian Concrete Journal* (Bombay), V. 30, No. 9, Sept. 1956, pp. 277-279

A prestressed concrete bridge, 16 spans of 110 ft each, had to be constructed at an average height of 70 ft above river bed. Superstructure consists of two I-shaped precast, prestressed girders connected by cast-in-place diaphragms and a cast-in-place deck slab. Girders were precast on shore and erected by launching on rollers from shore on temporary erection trusses.

Some slab-bridges and beam-bridges in prestressed concrete (Nogle pladeog bjaelkebroer i forspaendt beton)

C. Ostenfeld and E. Kalhauge, *Ingeniøren* (Copenhagen), V. 65, No. 44, Nov. 3, 1956, pp. 870-881

Reviewed by Jesper Strandgaard

Authors give summarized description of 22 smaller, simple span and continuous slab bridges and beam bridges built in Denmark, Norway, and Sweden from 1951-1956, all designed by C. Ostenfeld and W. Jenson, consulting engineers, Copenhagen, Denmark. A table gives basic geometrical and structural

A part of copyrighted Journal of the American Concrete Institute, V. 28, No. 11, May 1957, *Proceedings* V. 53. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

For those members that cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

data for all the bridges, with span lengths varying from 40 ft to about 120 ft.

General principles for arrangement of cables in simple and continuous spans are briefly analyzed with the conclusion that the simplest possible arrangement is usually the cheapest, in spite of an increased consumption of material. The illustrated article has an English summary.

Construction

Factory in concrete for the Pressed Steel Co., Ltd., Stratton, St. Margaret

D. H. New and R. Sanders, *The Structural Engineer* (London), V. 35, No. 2, Feb. 1957, pp. 69-80

A factory approximately 400 x 600 ft utilized a maximum of precast concrete. The longer span elements were prestressed, others reinforced. Precast work included columns, beams, purlins, wall panels and rafters for saw-tooth roofs. Structural steel crane girders were used.

Lift-slab construction

Commonwealth Engineer (Melbourne), V. 44, No. 5, Dec. 1956, pp. 149-151

Describes the design of an eight-story apartment building to be built by the lift-slab method at Elizabeth Bay, Sydney.

Monsanto House, London

Prefabrication (London), V. 3, No. 36, Oct. 1956, pp. 537-542

The building, 214 x 46 ft, two stories below ground and eight above, was designed to give maximum use of prefabricated precast concrete units from first to eighth floor. Construction consisted of cast-in-place columns with precast beams, floor units, stair flights, and wall panels. Wall panels were designed to serve as back-up.

Pumps and power buggies place concrete for Mackinac deck

Contractors and Engineers, V. 54, No. 1, Jan. 1957, pp. 94-98

Describes paving operation for Mackinac Straits Bridge, which will have a 6-in. concrete deck 54 ft wide, 10,247 ft long. Concrete was placed by pumpline on one end of bridge, being pumped as much as 1185 ft from machine to point of placement. On other end of bridge concrete buggies were used.

Walls for a six-story building erected in 4 days

Prefabrication (London), V. 4, No. 37, Nov. 1956, pp. 14-16

A six-story nurses' home utilized the sliding-form method to complete the structural walls in 4 days. First, four columns were constructed by conventional methods; the remaining five floors bear on the end corridor and partition walls, which were constructed by the sliding-form method.

Underground warehouse excavated in granite

C. D. Zernichow and H. Moyner, *Civil Engineering*, V. 27, No. 1, Jan. 1957, pp 50-53.

For an underground warehouse in Oslo, Norway, consisting of six levels, 633 ft long, with access tunnels, precast, vacuum processed elements were used for the wall panels and arch ceilings. Cast-in-place columns were also vacuum processed.

Concrete co-op will rise 40 stories

Engineering News-Record, V. 158, No. 6, Feb. 7, 1957, p. 89

A reinforced concrete apartment building in Argentina is said to be one of the world's largest reinforced concrete buildings. Overall dimensions are given. Building will include one tower 49 x 231 ft, rising 40 stories in height.

Factory at Dronfield, Yorkshire

Concrete and Contructional Engineering (London), V. 52, No. 1, Jan. 1957, pp. 47-50

Describes general concrete construction in a new tool factory including precast roof framing and framing for overhead cranes. Of special interest is a detailed illustration and description of the design of foundations for forging hammers.

Construction of a covered service reservoir in colloidal grout concrete

H. Seddon, *Journal*, Institution of Water Engineers, London, V. 10, 1956, pp. 20-50

This covered service reservoir was constructed in colloidal grout concrete because company desired to build reservoir with its own labor force, which would, from time to time, be withdrawn from the project to work on more urgent maintenance and repairs.

Colloidal grout concrete (a type of prepacked concrete) offered advantage that placing of coarse aggregate and steel could go ahead at any speed depending on labor force available, and then be grouted at a later date.

Base and sidewalls were one continuous slab without expansion or contraction joints. Reinforcement in the floor slab consisted of expanded steel sheets. Beam and slab roof construction utilized asbestos-cement corrugated sheets as formwork which then became a permanent part of the slab. Colloidal grout concrete is discussed in some detail.

Supplement to 1955 edition of ASTM standards in building codes

American Society for Testing Materials, Philadelphia, 1956, 224 pp., $2.75

The 1956 supplement to the 1955 compilation of *ASTM Standards in Building Codes* includes revisions and new ASTM standards for construction materials that have been accepted since publication of the 1955 edition. Seven new specifications and two new methods of test are included. Twenty-one standards and tentatives with extensive revisions are published in their entirety and 63 standards and tentatives have small revisions completely described to bring the 1955 compilation up-to-date.

The 1955 compilation and this supplement together cover the full range of construction materials including structural and reinforcing steel; steel, cast iron, and copper pipe; building units; bituminous roofing; cement; gypsum; ready-mixed concrete; and many other materials.

Dams

Thin arch dams (Les barrages en voute mince)

Jean Lombardi, F. Rouge, Lausanne, Switzerland, 1954, 163 pp., 28.70 F. (Swiss); Dunod, Paris, 1956, 2500 F. (French) (cloth), 2250 F. (stitched)

Reviewed by Aron L. Mirsky

Author considers thin arch dams as thin shells. On this basis, and with the aid of certain simplifications and by neglecting certain factors (a whole chapter is, however, devoted to evaluating the relative importance of these neglected factors), he develops a method of analysis of arch dams. The problem of torsion is treated fully, as is the relation of foundation and arch. A comparison of author's method with several classical and current methods is afforded by a numerical example (pp. 91-133).

A volume slim in format but *not* in content, recommended to all interested in design and analysis of dams.

Prestressed dam in Scotland

Concrete and Constructional Engineering (London), V. 51, No. 7, July 1956, pp. 419-423

A vertically prestressed dam was anchored approximately 27 ft deep in bedrock granite. High-strength alloy steel bars were used for the prestressing tendons.

Design

On the subject of self-supporting spiral stairs (in German)

H. S. Gedizli and W. Fuchssteiner, *Beton-und Stahlbetonbau* (Berlin), V. 50, No. 9, Sept. 1955, p. 247

Applied Mechanics Reviews Nov. 1956

A useful table to supplement a previous paper by Fuchssteiner ("Self-Supporting Spiral Stairs," *Beton-und Stahlbetonbau*, V. 49, No. 11, Nov. 1954, pp. 252-259).

On the problem of buckling of a hyperbolic paraboloidical shell loaded by its own weight

A. Ralston, *Journal of Mathematics and Physics*, V. 35, No. 1, Apr. 1956, pp. 53-59

Applied Mechanics Reviews Nov. 1956 (Costakos)

Shell is chosen so that its four edges are straight lines. Edges are assumed to be members that are rigid in direction of their axes, have negligible bending resistance in planes tangent to the shell, and whose support is moment free. By adding to these boundary conditions one which assumes that the resultant shear which each edge stiffener applies is same in both buckled and unbuckled state, problem is shown to be same as for a flat rectangular plate.

Buckling curves are given only for a square plate because of the numerous calculations required. These curves are similar to other buckling curves except that two systems are necessary according to an even or odd criterion for solution of the differential equation.

Designing of bending up of cables in prestressed concrete beams (in Polish)

J. Mames, *Inzynieria i Budownictwo* (Warsaw), No. 5, 1955, pp. 151-157

Polish Technical Abstracts No. 2 (22), 1956

Presented as a simplification of ordinary methods for designing bent-up cables in prestressed beams. Author indicates that method used by Guyon to determine bend-up point necessary is erroneous.

Torsion in reinforced and prestressed concrete beams

H. J. Cowan, *Journal*, Institution of Engineers, Australia (Sydney), V. 28, No. 9, Sept. 1956, pp. 235-240

Twenty-three references are given, including at least 13 involving research, and the principal results briefly described. A design procedure similar to that established for the design of sections subject to both bending moments and shear forces is suggested. Recommendations for plain concrete, reinforced concrete, prestressed concrete, and concrete-encased structural steel are included.

The problem of sway in moment distribution: General treatment using concept of virtual work

D. H. Clyde, *Engineering* (London), V. 181, No. 4709, June 8, 1956, pp. 457-459

Reviewed by Aron L. Mirsky

Correction for sway is stated to be necessary when number of independent equations of statics relating end moments of members exceeds number of joints at which moments are distributed. Paper demonstrates that sway corrections may be formally treated in a similar manner to basic joint-balancing procedure, the preliminary work involved in the analysis thus being simplified. A two-bay gable frame with pinned ends is analyzed as an example.

A consideration on the safety of structure by plastic and statistical theory

I. Konishi and M. Shinozuka, *Proceedings of Fifth Japan National Congress for Applied Mechanics*, Mar. 1956, pp. 83-86

Authors examine failure of a beam subjected to a single rolling load prescribed by probability density function. Two types of failure are visualized: static collapse and incremental collapse.

Stress distribution in box and corrugated sections (of reinforced concrete members) with small eccentricity (in Serbian)

Djordje Lazarevic, *Naše Gradevinarstvo* (Belgrade), V. 10, No. 9, 1956, pp. 1305-1317

Reviewed by J. J. Polivka

Such sections used in precast members and also in large elements and structural parts cast in place should be designed for maximum bearing capacity. Author presents simplified methods of design. For ratios between the tensile and compressive stress not exceeding 1:4 it is possible to determine minimum depth of the member without ignoring the tension zone. Designs for small and large eccentricities are investigated and numerical examples worked out.

Discussion on load tests on a three-story reinforced concrete building in Johannesburg

The Structural Engineer (London), V. 34, No. 10, Oct. 1956, pp. 360-369

Discussers questioned actual working stresses used in calculation of design loads, methods of designing web reinforcement for shear, comparison of stresses measured in main reinforcement and in web reinforcement with calculated values, safety precautions during gravity loading to failure, and offered general comments on system of loading and behavior of slabs under failure load. (See "Current Reviews," ACI Journal, Feb. 1956, *Proc.* V. 52, p. 702 for review of original article.)

Limit analysis for nonproportional loading (in Italian)

V. Franciosi, *Ricerca* (Naples), V. 5, No. 3, 1954, pp. 23-28

Applied Mechanics Reviews Oct. 1956 (Prager)

For expository reasons, determination of load factor of a structure by limit analysis is usually discussed as if a common load factor applied to all loads (see, however, H. J. Greenberg and W. Prager, *Transactions*, ASCE, V. 117, 1952, pp. 1481-1482). Present author points out that these methods are readily extended to the case where load factor is applied only to some loads while the intensities of the other loads are fixed.

On the improvement of stress distribution of a braced fixed arch by prestressing and its relation to the process of arch closure

E. Murakami, *Proceedings of Fifth Japan National Congress for Applied Mechanics*, Mar. 1956, pp. 221-224

Describes calculation and execution of arch closure for flat fixed arch truss.

Plane elastic systems—Method of the principal redundant (Sistemi elastici piani—Metodo dell' incognita principale)

Attilio Linari, Pellerano-Del Gaudio, Naples, 1956, V. I, 242 pp., 2000L; V. II, 144 pp. 1000L
Reviewed by J. J. Polivka

Author's method refers to simplified analysis of all problems pertaining to continuous beams, rectangular and polygonal frames, multiple-story framework, and other highly statically indeterminate structures (like Vierendeel trusses) with constant and variable moments of inertia and for any type of loading. Presented method is exact (no approximations like methods of moment distribution) and—by using tabulated values in V. II—amazingly simple and time saving. Author introduces "principal redundants" the number of which is reduced to one in all structural problems relating to continuous beams of any number of spans and to hemisymmetrical Vierendeel trusses (with respect to horizontal axis). Tables furnish exact values of elastic rotations at any support, deflections and bending moments in any section as function of the bending moment (or rotation) at end support and with respect to different elastic weight (stiffness) of individual members, their restraining moments (fixed-end moments), and under consideration of possible yielding at supports. Table values permit also determining directly the location of the fixed points (conjugate points) and also the section of maximum moment in any span or member and influence lines of the aforementioned values. Advantages of the method are demonstrated on many examples. Numerous references are cited.

Reviewer believes that important merit of Linari's book is revival of old principles of analyzing hyperstatic structures in exact and simple way; these, unfortunately have been overlooked during the last decades.

Precast shell elements of large roof structures. Part II

Dan D. Niculescu, *Industria Constructiilor si a Materialelor de Constructii* (Bucharest), V. 7, No. 5, May 1956, pp. 265-270
Reviewed by J. J. Polivka

Presentation of simplified membrane theory applied to precast shell elements with double curvature. Results of the analysis are compared with loading tests of a section 21 ft wide, spanning 47 ft, as already described in Part I (No. 11, 1955, p. 574). Fissures and cracks were observed and recorded under various concentrated loadings.

Stresses in single-span deep beams

F. E. Archer and E. M. Kitchen, *Australian Journal of Applied Science* (Melbourne), V. 7, No. 4, Dec. 1956, pp. 314-326

Presents direct method for determining stress distribution in single-span beams. Solution obtained is in two parts: (1) being equivalent to expressions according to elementary beam theory; and (2) representing correction to be made to these stresses. Several numerical examples are carried out and a comparison to a photoelastic solution is given.

Impact of transverse cranes and suggested impact loads for crane runway girder design

M. Takeuchi, *Proceedings of Fifth Japan National Congress for Applied Mechanics*, Mar. 1956, pp 225-226

Theoretical analysis of impact—horizontally, vertically, and in the axial direction—of a crane girder. Author arrives at the following percentages: (1) vertical 10 percent; (2) horizontal 2.5 percent, and (3) axial 20 percent.

Analysis of a parabolic arch of uniform thickness

V. A. Morgan, *Concrete and Constructional Engineering* (London), V. 51, No. 4, Apr. 1956, pp. 335-343
Applied Mechanics Reviews Oct. 1956 (Finlay)

General equations are derived which can be used in design of arches of any rise-span ratio. The most usual case, where rise is one-quarter of span, is used here, and stiffness ratios, carry-over factors, arch thrust are derived for this ratio to simplify the calculations. Properties of arches with other rise-span ratios can be calculated from equations derived.

Effect of shrinkage and creep of concrete on cracked reinforced concrete girders (in German)

L. Kollar, *Acta Technica*, Academiae Scientiarum Hungaricae, Budapest, V. 14, No. 3-4, 1956, pp. 293-311

Ceramic Abstracts Oct. 1956 (Hartenheim)

A theoretical investigation is made of changes in stress distribution and in location of neutral axis so that, in practice, composition and strength of the concrete can be adapted to these conditions.

Grid bridge design

Louis Baloo, *Civil Engineering*, V. 27, No. 1, Jan. 1957, pp. 40-45

Presents method for consideration of transverse framing stiffness in distributing live loads to girders of a bridge. Claims that consideration of transverse stiffness contributed by slab and diaphragms in bridge design specifications of AASHO may be improved by rational method of considering transverse stiffness presented.

Statically indeterminate frames: the two basic approaches to analysis

T. M. Charlton, *Engineering* (London), V. 182, No. 4738, Dec. 28, 1956, pp. 822-823.

Reviewed by Aron L. Mirsky

Discussion of the equilibrium and compatibility methods and their relative merits in analysis of various types of redundant structures.

Case of the overweight skeleton

Architectural Forum, V. 106, No. 4, Apr. 1957, pp. 156-159

Discusses and compares ultimate strength design for reinforced concrete and plastic design for structural steel stating that these two new concepts in structural design are now ready for architectural application. Describes the contributions of Charles S. Whitney, Vernon P. Jenson, and J. F. Baker (England) to the development of ultimate strength and plastic methods of design.

Hyperbolic paraboloid roof is cheaper

Engineering News-Record, V. 158, No. 15, Apr. 11, 1957, p. 51

A hyperbolic paraboloid thin shell concrete roof was used for a Kansas City water reservoir. The design was the least expensive of four alternates, including precast channel slabs, precast prestressed channel slabs, and conventional flat slabs. The 415-ft diameter reservoir will use fifty-two 46-ft 10 in. square shells in the roof. Columns are spaced 45 ft 10 in. each way.

Analysis of bowstring arches

F. H. Turner, *Concrete and Constructional Engineering* (London), V. 52, No. 2, Feb. 1957, pp. 61-72

Presents a method for solution of reinforced concrete bowstring arches. Tables are given for the case of a parabolic arch in which the section is varied as a function of the slope. The author states that a direct analysis would require determination of at least ten redundants, whereas his solution involves only four.

Materials

Influence of inorganic chlorides and sulfates on the capillary water absorption of cement mortar (Einfluss anorganischer Chloride und Sulfate auf die kapillare Wasserabsorption von Zementmörtel)

G. Tsuneyama, *Zement-Kalk-Gips* (Wiesbaden), V. 9, No. 11, Nov. 1956, pp. 473-476

Author's Summary

Capillary absorption of 1:3 cement mortars to which various chlorides and sulfates amounting to 1 and 2 percent by weight of cement had been added was compared with absorption by mortars containing no admixture, the measurements being carried out under a pressure of about 3 mm water gauge in an apparatus invented by author. Among salts investigated, NaCl and KCl are found to produce the greatest decrease in absorption. There is, at the same time, an increase in flexural strength, whereas compressive strength is reduced. Ratio of absorption to evaporation is almost equal to unity in case of test specimens containing no admixture; it is less than unity if salts have been added which reduce absorption.

Testing the resistance of cements to sulfates (in German)

F. W. Locher, *Zement-Kalk-Gips* (Wiesbaden), V. 9, No. 5, 1956, pp. 204-210

Ceramic Abstracts Nov. 1956 (Hartenheim)

The aluminate component in cement forms, with alkali and $CaSO_4$ solution, ettringite,

$3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 32H_2O$, or the phase having only $12H_2O$, which is harmful to volume stability of the hardened cement or concrete. With Mg sulfate, the aluminates form Mg hydroxide and gypsum, which may also be harmful. In this process tricalcium silicate is attacked much more rapidly than dicalcium silicate, and damage appears as a loosening of texture and expansion phenomena caused by crystal growth or hydrostatic crystallization pressure. Different methods of testing sulfate resistance are described.

Mineralogical information for the study of adhesion between hydrated constituents of cements and the bound materials (Contribution mineralogique à l'etude de l'adhérence entre les constituants hydratés des ciments et les matériaux enrobés)

J. Farran, *Revue des Matériaux de Construction* (Paris), No. 490-491, July-Aug. 1956, p. 155; No. 492, Sept. 1956, pp. 191-209

Reviewed by Phillip L. Melville

A thorough mineralogical study of the bond between the aggregates and the binder by microscopic inspection of dyed thin sections of hardened concrete. The following cements were investigated: portland, slag, sulfate, and alumina. The following aggregates were investigated: calcite, dolomite, quartz, flint, feldspar, mica, and anhydrite. Three factors were taken into consideration: surface condition of the particles, curing and aging, and amount of mixing water. Best adhesion was observed with calcareous aggregates and portland cement. Bond was found very poor for micas. The interface was found constituted of hydrated monocalcic aluminate with alumina cements of portlandite for portland cements, of portlandite and aluminate and sulfoaluminate of calcium for slag cements, of sulfoaluminate of calcium for sulfate cements. It is also concluded that limestone is not an inert aggregate. Twenty-five references are addended.

Masonry cements

Charles E. Wuerpel, *Proceedings Third International Symposium of Chemistry of Cement*, Cement and Concrete Association, London, 1954, pp. 633-679

Ceramic Abstracts Oct. 1956 (Newman)

History of modern masonry cements is reviewed. Present-day cement is usually a finely interground homogeneous mixture of portland cement clinker, limestone, gypsum and air-entraining agent. The existing specifications for masonry cements are very loose in performance requirements. This weakness has led to expensive failures in masonry construction, an example of which is described with tests and photographs. It is recommended that tests for the following properties be added to existing specifications: autoclave soundness, workability retention, plasticity, air content, and loose bulk density. Bond tests, although desirable, do not seem feasible for inclusion at present. Tests for above properties and others were made by four cooperating laboratories using 16 masonry cements. Physical properties of 20 presently manufactured masonry cements were tested in the author's laboratory. Test results are given and discussed. A description is given of the apparatus and methods used for testing plasticity, workability retention, bond strength, and stiffening index. (See also "Current Reviews," ACI Journal, May 1955, *Proc.* V. 51, p. 954.)

Expansive cements

H. Lafuma, *Proceedings Third International Symposium of Chemistry of Cement*, Cement and Concrete Association, London, 1954, pp. 581-597

Ceramic Abstracts Oct. 1956 (Newman)

Expansive cement is added to portland cement to counteract its drying shrinkage and to produce a nonshrinking cement. Expansive agent is a calcium sulfoaluminate cement consisting largely of (1) anhydrous calcium sulfate, (2) calcium aluminates, particularly $5CaO \cdot 3Al_2O_3$, and (3) β-$2CaO \cdot SiO_2$. Granulated blast-furnace slag is added as a stabilizing agent to control time of occurrence, duration, and extent of expansion. Composition, fineness, and relative proportions of the three constituents must be considered in obtaining required expansion. (See also "Current Reviews," ACI Journal, May 1955, *Proc.* V. 51, p. 954.)

Observed unevenness in the burning of portland cement clinkers (Des irregularités observées dans la cuisson des clinkers à ciment portland)

A. Virella, *Revue des Matériaux de Construction* (Paris), No. 490-491, July-Aug. 1956, pp. 173-177

Reviewed by Phillip L. Melville

Unevenness in clinker has often been blamed on the fuel although it is observed

with cinderless fuels and does not correspond with the quality of the new materials. The common increase in silica and decrease in lime appears to be caused by segregation of the components in the raw materials with the finest (calcareous) carried off by the draft. It is concluded that fine grinding of the raw clinker, combined with rigorous agitation, will largely prevent this segregation. Twenty seven references are addended.

Bond with reinforcing steel

H. J. Gilkey, *Special Technical Publication* No. 169, ASTM, 1955, pp. 143-162 (Reprint available from Iowa State College Engineering Experiment Station)

Reviews research on the bond properties of various types of reinforcement, development of specifications, both for bars and bond, and discusses some of conclusions accepted in present practice. Considers effects of various load conditions, types of concrete and position in concrete on bond with reinforcement. States that standard method of test developed by ACI Committee 208 is necessary both as a check-up procedure and as a standing hurdle to be surmounted by any form of bar purporting to qualify as one of the acceptable types of deformed bar reinforcement. Includes 77 selected references as a bibliography.

Present position as regards the testing of admixtures to concrete (Uber den heutigen Stand der Prüfung von Betonzusatzmitteln)

K. Walz, *Zement-Kalk-Gips* (Wiesbaden), V. 9, No. 10, Oct. 1956, pp. 437-440

Reviewed by H. H. Werner

Standard testing methods, as required by the German building code, for plasticisers and air-entraining agents are outlined and discussed. Experience to-date has shown this uniform testing to have real significance and to have promoted correct and efficient use.

Production and mode of action of molecular silicone films on materials of a silicate nature (Erzeugung und Wirkungsweise molekularer Siliconfilme auf silikatischen Werkstoffen)

W. Noll und H. Weissbach, *Zement-Kalk-Gips* (Wiesbaden), V. 9, No. 11, Nov. 1956, pp. 476-485

Reviewed by H. H. Werner

Silicones are an eminently suitable aid in preparing and improving industrial materials of a silicate nature by their possibility of rendering surfaces of such materials water-repellent. Technically available preparations for this purpose are discussed, their modes of action are tentatively explained, and practical applications are dealt with. Nature of problem and its solution are closely bound up with type of silicate and, furthermore, with question whether effect to be achieved is purely a surface effect (glass) or whether a certain action in depth is aimed at (building materials). Effects that can be achieved are demonstrated by means of measurements on silicone-treated glass (contact angle method) and on silicone-impregnated building materials (permeability to air and to water, water absorption.) Depth of penetration for materials containing cement is less than for materials which contain no cement and this is connected with the extent to which binding of free lime (carbonization) has progressed (i.e., with age of the material concerned). Resinous silicones have widest range of applicability in sphere of building materials. Further effects resulting from treating surfaces of a silicate nature with silicones are briefly indicated.

Lightweight aggregate concrete—gravel, clinker and lightweight aggregate concrete blocks (Béton d'agrégats légers—blocs de béton de gravillons, de machefer, d'agrégats légers)

CSTB Staff, *Cahiers*, Centre Scientifique et Technique du Batiment (Paris), *Bulletin* No. 218 (no date), pp. 1-22

Reviewed by Edmund A. Pratt

Contains a technical document (proposed standard) defining pozzolan and execution and application of pozzolanic lightweight concrete; a similar technical document defining expanded slag and execution and application of expanded slag lightweight concrete; specifications for sand and gravel concrete blocks for walls and partitions; specifications for pozzolanic or expanded slag lightweight concrete blocks for walls and partitions; and specifications for clinker concrete blocks for walls and partitions. Acceptance conditions are annexed to the "technical documents" and general manufacturing recommendations and general conditions of application are appended to the specifications.

Testing of cement by method of plastic mortar (in Serbian)

VLASTIMIR TUFEGDZIC, *Naše Gradevinarstvo* (Belgrade), Part I, V. 10, No. 6, 1956, pp. 814-818; Part II, V. 10, No. 7, 1956, pp. 990-994

Reviewed by J. J. POLIVKA

Yugoslav cements have been generally judged by compressive strength at 28 days of cement mortar mixed 1:3 in 7.07-cm cubes. New testing method was introduced, using prismatic specimens of plastic mortar, 4 x 4 x 16 cm, with admixture of normal sand and with constant W/C ratio of 0.44. Author presents a thorough report on new standard specifications and test methods. In Part I various testing methods employed from beginning of this century are discussed. New methods on the basis of plastic mortar introduced in Yugoslavia with important contributions by Jelisavčic and Ristić are described and various characteristics of four typical Yugoslav cements are examined without being compared with those resulting from tests with dry mortar. In Part II author presents results of tests in compression and bending with both cubic and prismatic samples and concludes with the emphasis that mechanical tests alone should not be considered as sufficient for evaluation of cements; chemical properties should also be thoroughly investigated.

Direct method of determining absorption and specific gravity of aggregates

EDWIN L. SAXER, *Rock Products*, V. 59, No. 5, May 1956, pp. 78, 79-87

CERAMIC ABSTRACTS Oct. 1956 (Barbour)

Describes an improved method for obtaining absorption by plotting a rate of absorption; the difficulty in bringing the material to a surface dry state after inundation is eliminated as absorption is constant. A sample data sheet is included.

Numerical evaluation of the grain size of concrete aggregates (Zahlenmässige Bewertung der Kornzusammensetzung von Betonzuschlagstoffen)

S. POPOVICS, *Acta Technica*, Academiae Scientiarum Hungaricae, Budapest, V. 13, No. 1-2, 1955, pp. 93-114

Reviewed by MILOS POLIVKA

Abrams' fineness modulus is roughly proportional to the average size of aggregates. Tests proved, however, that the theory of fineness cannot be applied to all gradings and that also other conditions of aggregates are of importance, e.g., proper distribution of gradation. Author discusses the well known graphical method, investigations of Hummel, tests by Palotas and research by Kolmogorow, and introduces general formula:

$$30.1\, m = 100 \log 10\, D - 0.4343 \int_{0.1}^{D} \frac{f(d)}{d}\, dd$$

in which m is the fineness modulus, D the maximum grain size in mm, and $f(d)$ the function (equation) of gradation curve expressing the relation between the grain size d and grains of smaller diameter than d (percentage). It has been found by tests that the optimal value of the integral from standpoint of compressive strength of concrete can be expressed as

$$\int_{0.1}^{D} \frac{f(d)}{d}\, dd = 45.88 \log D + 165.07 - 0.1940\, c$$

in which c designates cement content in kg per cu m. Palotas test results show that the most favorable fineness modulus (in sq in. per kg) can be achieved by the following relation:

$$m_o = 2.66 \log D + 0.84 + 0.028\, c$$

It is concluded that the specific surface is characteristic of the spread of distribution and that the fineness modulus yields a dependable basis for the evaluation of continuous grain structure, assuring the necessary degree of spread. In case of discontinuous granulometry it is necessary to introduce a new numerical characteristic of the grain structure, i.e., specific fictive surface. The paper presents numerical and graphical methods of determining such specific fictive surface. Results of tests are shown in ten diagrams (e.g., relation between strength and specific surface for various fineness, gradation for constant fineness modulus and equal specific surface).

Studies relating to the testing of fly ash for use in concrete

R. H. Brink and W. J. Halstead, *Public Roads*, V. 29, No. 6, Feb. 1957, pp. 121-141

Effect of fly ash as a cement replacement on strength of mortars is used as an index of its effect on the strength of concrete. Its effect on expansion due to alkali-aggregate reaction is also considered. Thirty-four fly ashes from 19 different sources were examined. Results indicate generally that important factors are carbon content of fly ash, fineness of fly ash (passing No. 325 sieve), water requirement, and particular cement used. Property of reducing expansion due to alkali-aggregate reaction was found to vary with different fly ashes.

An evaluation of crushers and their relation to the particle shape of blast furnace slag and stone

Heinrich Kahlhofer, Alfred Send, and Herbert Kaiser, *Stahl und Eisen* (Duisburg), V. 76, 1956, pp. 957-964

Reviewed by E. W. Bauman

Several interesting references to work of others dealing with particle shape of aggregates are included: K. Walz, W. Pickel, G. Rothfuchs, F. Keil, D. Rösslein, and K. Schulze describe methods for measuring and evaluating flat and elongated particles using the weight per cu m in the process.

Work-hardened twisted reinforcing steel

Kurt Billig, *Civil Engineering and Public Works Review*, (London), V. 52, No. 607, Jan. 1957, pp. 57-59

Reviews and compares properties of three commonly available types of work-hardened reinforcing steel in use on the continent—Isteg, Tor, and square grip bars. Comparative data are presented on strength properties, hardness, modulus of elasticity, and elongation for these various types of cold work reinforcement. (Part 1 of a series of papers on this subject.)

Slag cements

F. Keil, *Proceedings Third International Symposium of Chemistry of Cement*, Cement and Concrete Association, London, 1954, pp. 530-580

Ceramic Abstracts Oct. 1956 (Newman)

Slag-portland cement, portland cement-slag, and slag-sulfate cements are discussed. These cements contain as an essential element granulated blast-furnace slag in chemically unchanged form, the first-named component being in each case the principal constituent. Chemical and physical methods of evaluating a slag for use in a cement are discussed, and examples are given. Small-specimen compressive and bending tests are described, and the results of such tests are compared with the results obtained with specimens of conventional size. Methods of determining slag content of a slag cement are described. (See also "Current Reviews," ACI Journal, May 1955, *Proc.* V. 51, p. 954.)

How to determine the value of a cement (Comment determiner la valeur d'un ciment)

H. Manche, *Revue des Matériaux de Construction* (Paris), No. 486, Mar. 1956, pp. 55-60

Reviewed by Phillip L. Melville

A systematic comparison of French and Belgian testing methods. The latter compact test cubes 50 sq cm in section by dropping a hammer weighing 3 kg from 50 cm 150 times. The former uses a plastic mix which gives advantage to portland over slag cements and to fine over coarse ground cements. The correlation between cement testing and concrete quality is poor from a quantitative point of view, especially when specimens made from plastic mixes are used.

On the determination of the hydraulic power of pozzolans (Sur la détermination de l'activité hydraulique des pouzzolanes)

Steopoe, *Revue des Matériaux de Construction* (Paris), No. 492, Sept. 1956, pp. 210-212

Reviewed by Phillip L. Melville

A method is discussed to measure pozzolanic activity by evaluating the reaction of pozzolans with hydrated lime at room temperature in a CO_2-free atmosphere. Ten references are listed.

Pavements

Sawed joints in concrete pavements: Progress and problems

E. J. Coppage, Jr., *Bulletin* No. 132, Highway Research Board, Dec. 1956, p. 31-34

Discusses use of sawed joints in highway paving and problems they create in design, curing, and construction.

Modernizing a Texas highway with concrete

A. L. Chollar, *Proceedings*, ASCE, V. 82, HW3, Oct. 1956, pp. 1074-1 to 1074-7

Reviews design and construction procedures for concrete resurfacing over an existing 16-ft pavement 34 years old. Construction details of longitudinal and transverse jointing procedures for the new 24-ft pavement are described.

Quality concrete for highway construction with central mixing plant

Glenway Maxon, *Bulletin* No. 132, Highway Research Board, Dec. 1956, pp. 1-10

Describes and pictures new equipment for highway construction, permitting better control of quality. Discusses newly developed tests for uniformity of mix, consistency, and cement content in fresh concrete.

Slip-form paver does complete mechanized job

Contractors and Engineers, V. 54, No. 1, Jan. 1957, pp. 116-118

A slip-form paver was used in construction of an 8 in. thick, 24 ft wide, 12 mile long highway in Colorado under specification requirements of ⅛-in. deviation in 10 ft. Trailing slip-forms 60 ft long were used. Transverse joints were cut by sawing. Curing was accomplished by white-pigmented curing compound.

Concrete road construction today

D. R. Sharp, *Civil Engineering and Public Works Review* (London), V. 51, No. 606, Dec. 1956, pp. 1363-1365

Summarizes designs, specifications, and construction methods now in use in British paving practice. Includes bibliography of 18 references; first part of a series of papers on this subject.

Precast Concrete

Precast arches form gym roof

Engineering News-Record, V. 158, No. 5, Jan. 31, 1957, pp. 45-47

A California high school gymnasium consisting of precast, three-hinged concrete bents of 90-ft span, supports precast, cylindrical shell units. Shell units weigh 20 tons each, are 3 in. thick, 61 ft long, and 15 ft wide.

Method of precasting box beams, plates, etc., by using vibration and turning the forms

I. G. Contescu, *Industria Constructiilor si a Materialelor de Constructii* (Bucharest), V. 7, No. 5, May 1956, 276-278

Reviewed by J. J. Polivka

The method was used for precasting of 5000 U-shaped panels, 30 in. wide and 20 ft long, assembled into a roof structure spanning approximately 38 ft, two panels pitched and held in position by tie-rods with turn-buckles. Admixture of calcium chloride and proper vibration in double steel forms successively turned 180 deg made it possible to remove the forms after 48 hour.

Organization of mixing and precasting plants (in Rumanian)

Trustul 5 (In collaboration with other groups of technology), *Industria Constructiilor si a Materialelor de Constructii*, V. 7, No. 4, Apr. 1956, pp. 210-221

Reviewed by J. J. Polivka

Design of typical temporary and permanent installations of mixing equipment for concrete and mortar, for precasting and curing structural members, transportation and storage of materials, with the emphasis of advantages, especially of attained speed and good quality of products.

Precast system for factory extension

Prefabrication (London), V. 3, No. 36, Oct. 1956, pp. 550-552

In an almost wholly precast concrete four-story factory addition, interior columns are spaced 32 and 46 ft. Floors are 10 in. deep in the 32-ft spans and 15 in. deep in the 46-ft spans. All are designed for 120 lb per sq ft live load. Floors, columns, and beams are prestressed.

Precast concrete joists with and without hollow blocks system "KAT" (in Serbian)

Vlado Kamenarovic, *Naše Gradevinarstvo* (Belgrade), No. 7, 1956, pp. 1001-1004

Reviewed by J. J. Polivka

The reversed T-shaped beams support hollow blocks with spaces at the top to be grouted after installation. Floor finish is either linoleum on 1¼ in. asphalt underlayer or 2-in. tongue-and-groove boards on 2-in. insulating layer. Typical spacing of joists is 19 in. However, it can be reduced, or the joists jointed, in which case tapered spaces are filled with concrete with additional reinforcing.

Cast stone as a modern building material (Betonwerkstein im neuzeitlichen Bauschaffen)

HELLMUTH FRIESEN, Erasmusdruck Max Drause, Mainz, Germany, 95 pp.

A book devoted to notable applications of cast stone (precast concrete) in many countries. It includes some 16 beautiful color illustrations, 75 black-and-white photographs, and a number of detailed sketches illustrating connection devices and other details which were utilized. Quality and variety of illustrations should make this book an interesting addition to the files of the modern architect and of particular interest to cast stone producers.

Prestressed concrete screens for drive-in theaters

Constructional Review (Sydney), V. 29, No. 12, Dec. 1956, pp. 31-32

Drive-in theater screens were precast prestressed concrete panels, 3 in. thick (post-tensioned), with smooth steel trowel finish to form screen surface. Supporting framework was structural steel. Builders claimed that weight of concrete panels is advantageous in this type of structure which must be designed for overturning due to wind load. Design has been tested by an unexpected storm, producing winds of 75 mph during erection and so severe that scaffolds were destroyed by wind, forcing workers to take refuge on screen itself.

Floor elements double as service ducts

Constructional Review (Sydney), V. 29, No. 12, Dec. 1956, pp. 20-22

Precast concrete elements were made in form of inverted troughs for a laboratory building. Floors and roofs of the building consisted of precast trough-shaped units, supported by inverted T-shape cast-in-place girders. Water, gas, electricity, and waste water lines are all contained in troughs. Precast slabs were used between troughs to furnish floors and acoustic panels to fill soffit troughs, providing ceiling of alternately exposed concrete troughs and acoustic panels.

Precasting and prestressing of concrete trestle members done swiftly at special plant

E. T. WEATHERS, *Contractors and Engineers*, V. 54, No. 2, Feb. 1957, pp. 106-109

Describes plant and equipment used to precast piles up to 106 ft in length, deck girders, decks, slabs and sheet piles for the Galveston-Pelican Island Causeway. This project required 500 trestle piles, 528 deck girders, 168 railroad deck slabs, and 2000 sheet piles.

Building constructed of large concrete blocks and precast slabs (in Rumanian)

Industria Constructiilor si a Materialelor de Constructii (Bucharest), V. 7, No. 4, Apr. 1956, pp. 234-242
Reviewed by J. J. POLIVKA

Construction of the large four-story building, 50 ft x 180 ft, served simultaneously for experimenting with large wall blocks and precast floor slabs of various mixes with different admixtures; results are reported as to properties and economy. Also the organization on the construction site is described.

Use of large wall blocks for apartment and other buildings (in Rumanian)

I. STEFANESCU, S. GHEORGHITA and D. GEORGESCU, *Industria Constructiilor si a Materialelor de Constructii* (Bucharest), V. 7, No. 1, Jan. 1956, pp. 29-44
Reviewed by J. J. POLIVKA

Large concrete blocks and wall panels for bearing walls and partitions of various sizes up to 100 sq ft, and thickness from 4 to 12 in., hollow and other types, are thoroughly described and their manufacture and assembly reported, including special units for corners, intersecting walls, windows, and doors. References are made to Russian methods.

Prestressed Concrete

Prestressed concrete bridges with concentrated tendons (Ueber Spannbetonbruecken, vorzugsweise mit konzentrierten Spanngliedern)

F. LEONHARDT, *Zement und Beton* (Vienna), V. 1, No. 5, May 1956, pp. 3-15
Reviewed by FRITZ KRAMRISCH

Article presents review of well-known method, employed and promoted by author, for post-tensioning by means of tensioning strands assembled in concentrated bundles that are looped at the ends around half

cylindrical stress blocks. Advantages of method, especially for bridge design, are described. A number of interesting applications are given.

Some improvements that have been made in the course of various designs and constructions are as follows: arrangement of long straight stretches between short polygonal curvatures and use of neat cable boxes with paraffined guides and slide plates have brought down the friction coefficient for longer constructions to a value between 10 and 20 percent. Continuous girders with up to seven bays have been successfully stressed this way. End anchorage has been improved through greater spreading of strands in anchor block; this reduces local stresses and permits intermediate anchor cone to be filled with grout after stressing. Voids between cables and boxwalls are kept of the same magnitude to offer equal resistance to flow of grout. Bigger voids in intermediate anchor cones are prepacked with gravel to be reduced to the same size. Observation windows covered with plexiglass have been provided to control desired stretching of the cables and progress of grout. Special presses are used with a capacity of up to 600 tons and 12-in. movement. They are constructed so as to permit a certain amount of play perpendicular to direction of stress to prevent binding. Electrical pumps speed stressing operations. For larger constructions prestressing in stages permits early application of a partial prestressing force to prevent shrinkage stresses and cracks at a time when concrete has not yet developed its required strength for final prestressing. On very long bridges concrete was poured in sections and the joints between were filled only shortly before first prestressing was applied. Overlapping of anchor loops over supports increased prestressing force in area of negative moments.

A series of bridges is described using the "Leoba" system, including: the first prestressed concrete railroad bridge in Germany across the Neckar Canal near Heilbronn; Rosenstein highway bridge near Stuttgart; and prestressed continuous girder bridge built across the Danube at Untermarchtal, which has a prestressed girder 1230 ft long without joints.

Prestressed concrete—its design and construction (Spannbeton in Entwurf und Aüsfuhrung)

G. Kani, Verlag Konrad Wittwer, Stuttgart, 1955, 585 pp., 45 DM

Applied Mechanics Reviews Oct. 1956 (Brandeis)

Qualities of basic construction materials are explained only briefly, as they can be found in many other books. In contrast, a complete view is given for methods of calculation and dimensioning and construction principles. Author, having constructed large buildings of prestressed concrete himself, states general rules as well as his own thoughts in clear terms so that book is not only useful for consulting but also for study.

Prestressed precast units for factory extension

Constructional Review (Sydney), V. 29, No. 12, Dec. 1956, pp. 28-30

Prestressed floor units were used in a four-story English factory engineering department. Prestressed precast floor units consisted of precast hollow boxes with closed ends approximately 6 ft long x 15 in. wide. Solid end anchorage blocks, two units wide, received prestressing cables contained in joints 3 in. wide between prestressed beam units. Separate lines of units and longitudinal cables were positioned by precast diaphragm spacer blocks in longitudinal joints. Transverse joints, 2 in. wide, were filled with high-strength mortar.

Design and construction of prestressed concrete hangars at Helsinki Airport, Finland

F. W. Gifford and M. Janhunen, *The Structural Engineer* (London), V. 34, No. 11, Nov. 1956, pp. 415-427

A double hangar has clear openings of 244 ft at the doors and a depth of 150 ft. Twin main arches over the door openings, 265 ft long, support secondary girders of 150-ft span with cantilevers at 23 ft at the rear. Prestressed concrete block sections were used for main arches, post-tensioned in place. Secondary girders were precast post-tensioned. Roofing units were pre-tensioned, factory made. In addition to design data and illustrations of construction and erection sequences, test data is included on a beam accidentally damaged by fire.

Prestressed precast platform built in gulf

R. N. Bruce, Jr., *Civil Engineering*, V. 26, No. 7, July 1956, pp. 41-43

Describes design and construction of the first all-prestressed, all-prefabricated, reinforced concrete platform erected in the Gulf of Mexico. The platform, utilized in an offshore oil well installation, and which supports two 5000-bbl oil tanks on a prestressed deck, is supported on spun, prestressed hollow piles up to 192 ft long.

Properties of Concrete

Effect of entrained air on strength and durability of concrete with various sizes of aggregates

P. Klieger, *Bulletin* No. 128, Highway Research Board, 1956, pp. 1-19

Previous tests with natural sand and siliceous gravel indicated that for concretes with constant cement content and consistency and varying water-cement ratios, adequate resistance to freezing and thawing was secured when the air content of the mortar fraction was 9 ± 1 percent regardless of the maximum size of aggregate. Tests reported herein on concretes using crushed limestone for a coarse aggregate confirm results of earlier tests. Report also includes information on effect of entrained air on strength, resistance to salt scaling, volume change, and absorption of these concretes.

Considerations about internal stresses and strains in concrete (Betrachtungen über Gefügespannungen im Beton)

B. Henk, *Zement-Kalk-Gips* (Wiesbaden), V. 9, No. 3, Mar. 1956, pp. 111-119

Reviewed by H. H. Werner

Investigations of durability of concrete under influence of varying conditions of heat and humidity must consider that concrete consists not only of anisotropic substances but, indeed, of two components—cement stone and aggregate—which often differ considerably in elasticity, thermal expansion, and swelling properties. Internal stresses, referred to as structural stresses, must consequently develop on subjecting material to heating and cooling, wetting and drying. But also stresses caused by external loading will be influenced by the difference in elasticity between the two components of concrete.

Equations are given and evaluated whereby stresses and strains developing in structural element, i.e., a sphere of aggregate with surrounding shell of cement stone, can be calculated if components behave differently with regard to expansion or if there is an external hydrostatic pressure. For purpose of this treatment of problem it is assumed that concrete components in themselves are isotropic and that their properties are virtually independent of influences considered, viz. temperature and humidity.

Use of fly ash in concrete

Albert G. Timms and William E. Grieb, *Public Roads*, V. 29, No. 6, Feb. 1957, pp. 142-150

A study of properties of concrete in which fly ash was used as a replacement for part of cement. Fly ash from four widely used sources was employed. From each source fly ashes of varying fineness were used. Tests included flexural and compressive strengths, shrinkage, durability under freezing and thawing, and resistance to attack by calcium chloride used for ice removal. Results showed in general that strengths at ages less than 1 year were less than those of plain concrete. Resistance to freezing and thawing was not improved. Drying shrinkage was reduced. All substitutions of fly ash for portland cement adversely affected resistance of both air-entrained and non-air-entrained concrete to attack by calcium chloride.

Maturity and strength of concrete

J. M. Plowman, *Magazine of Concrete Research* (London), V. 8, No. 22, Mar. 1956, pp. 13-22

Examines a relationship between concrete strength and maturity. Establishes a datum temperature for strength-maturity calculations. Develops a law relating the strength at any maturity to the strength at a given maturity. Strengths are expressed as percentage of that obtained at a maturity of 35,600 deg-F-hr (28 days at 64 F). The relationship is:

Percentage of strength at 35,600 deg-F-hr = A + B $\log_{10}$ (maturity in deg-F-hr/1000).

Preparation of concrete test samples (in Serbian)

Z. Miladinovic, *Nale Gradevinarstvo* (Belgrade), No. 7, 1956, pp. 985-987

Reviewed by J. J. Polivka

Typical sizes and shapes of concrete samples for testing are discussed and some of their inconveniences emphasized, especially effect of their preparation on compactness. Author proposes a new method of compacting the samples for uniform comparison.

Use of the Kelly ball for field measurement of concrete consistency

William E. Gries and Robert A. Mabr, Jr., *Bulletin* No. 132, Highway Research Board, Dec. 1956, pp. 22-30

Presents data of field tests comparing results of test measurements by the Kelly ball to the slump test for consistency of concrete.

Curing methods and duration studies of pre-tensioned units

A. M. Ozell and W. D. Givens, *Bulletin* No. 82, Florida Engineering and Industrial Experiment Station, University of Florida, Aug. 1956, 16 pp.

Presents research on the results of field and laboratory curing. Their effects upon concrete strength, both compressive and flexural, bond properties measured by slip of pretensioned strands, and modulus of elasticity at early ages. Concludes that usual methods of steam curing employed in Florida prestressed concrete plants develop sufficient concrete strength to permit stress release in 2 days, as compared to 3 days for wet-burlap-cured products simulating standard laboratory curing.

Interdependence of cement, aggregates, and air-entraining agent on the resistance to freezing and thawing of concrete

J. F. Orth, J. M. Blondel and J. A. Berrod, *Report* No. 9, (English translation), Fifth Congress on Large Dams, International Commission on Large Dams, Paris, 1955, 14 pp. Copies available from Région d'Équipment Alpes II, Electricité de France, 3 & 5 Rue Ronde, Chambéry, Savoie France.

A systematic study of possible combinations of air-entraining agents, aggregates, and cements was made. This report is concerned with concrete passing through a screen of 10-mm mesh with a cement ratio of 200 kg per cu m.

Composition and properties of concrete

G. E. Troxell and H. E. Davis, McGraw-Hill Book Co., Inc., New York, 1956, 434 pp., $7.75

An ideal text book for an undergraduate course in plain concrete. Scope covers concrete materials, proportioning, concrete production, placing, forming, curing, and testing. Functions of a concrete inspector or construction engineer are particularly well treated. An entire section of the book is devoted to selective laboratory tests to illustrate principles connected with use of cement and aggregates in concrete, with many numerical examples and lists of questions.

Disintegration of field concrete

G. M. Idorn, *Progress Report* No. 1, Danish Committee on Alkali Reactions in Concrete, Copenhagen, 39 pp.

Author's Summary

A review is given of typical evidences of field failure of concrete in Denmark. Qualitative remarks are made concerning the mechanism of alkali reaction, freezing and thawing, and sea water attack, and on the visible effects of these agents on the concrete. Observed evidences of failure are discussed on this basis.

It is concluded that safe interpretation of surface evidences, e.g. fracturing, will frequently not be possible if based only on field inspection. Such examination however should be considered the basis for a more detailed approach, e.g., by means of drilled cores. Reference is given to a forthcoming report dealing with examination of cores.

Sands and cements affect drying and shrinkage cracks

E. L. Howard and J. Ida, *Rock Products*, V. 59, No. 10, Oct. 1956, pp. 226-232

Studies showing effect of ten different sands and different cements available in California area on amount of drying and shrinkage cracks developed. It was found that each sand and each cement had its own crack pattern. Case histories from available job files confirmed conclusion that certain cement-sand combinations were more troublesome than others. All test panels moist cured, showed no cracks at any time. Panels subjected to drying air had cracking tendencies in proportion to the loss of moisture. Scope of this study did not include a method for testing characteristics of cement or sand that causes water to stay in or leave the mortar.

Osmotic method of increasing concrete strength

W. Pogany, *Civil Engineering and Public Works Review* (London), V. 51, No. 606, Dec. 1956, pp. 1349-1351

Notes that concrete strengths are theoretically dependent 90 percent upon porosity which is determined by water-cement ratio. Proposes method for reducing water-cement ratio. Proposed method involves treatment of fresh concrete by electric current. It is noted that temperature rise due to use of electricity should not exceed 50 C to prevent evaporation. Only direct current can be used since alternating current produces heat which causes too rapid evaporation. Simultaneous vibration is required also to minimize pores from which water has been removed. No experimental results are mentioned.

The water bound by cement (Das vom Zement gebundene Wasser)

R. Hayden, *Zement-Kalk-Gips* (Wiesbaden), V. 9, No. 3, Mar. 1956, pp. 120-122

Author's Summary

It is shown that nonbound water can be removed from hydrating cement by boiling with alcohol; state of hydration process can therefore be determined at any given moment.

With the aid of some examples it is demonstrated that the new method of investigation enables numerical data to be obtained in many cases where it has hitherto only been possible to make conjectures.

Accelerated testing of concrete

Constructional Review (Sydney), V. 29, No. 10, Oct. 1956, p. 35

Reports a technique for predicting compressive strength of concrete by test within a few hours after specimens have been made. Method consists of heating test specimens in an oven to a temperature just below boiling point beginning ½ hr after mixing; the temperature is raised in 1 hr and kept constant for an additional 5 hr. The test is completed ½ hr after heating ceases, a total of 7 hr. Covered steel molds retain the original mixing water for curing. There appear to be indications that the precise heating procedure need not be exactly the same but that different equally serviceable curves can be obtained by other standard procedures, perhaps 185 F instead of 200 F.

Effect of soluble salts and lime-cement mortar in masonry (in German)

K. Bergmann, *Ziegelindustrie* (Wiesbaden), V. 7, No. 13, 1954, pp. 542-544; abstracted in *Journal of Applied Chemistry* (London), V. 5, No. 5, 1955, p. 708

Ceramic Abstracts Nov. 1956

Experiments on causes of efflorescence are described. Tests show that brick containing only a small amount of soluble salts and having no tendency to effloresce as single brick can show considerable efflorescence after being built up in a wall. Single brick should be tested for efflorescence tendencies, and a suitable mortar should be used to avoid efflorescence.

Control in mixing and placing concrete

R. H. H. Kirkham, *Paper* No. 17, Public Works and Municipal Services Congress (England), Nov. 14, 1956, 12 pp.

Describes field control measures used in paving in Great Britain, particular emphasis being placed on control of compaction and surface uniformity during finishing. Utilizes compacting factor apparatus for field control of workability and cores cut in fresh concrete to determine both degree of uniformity in spreading and amount of surcharge required to obtain full compaction.

Recent research on the vibration of concrete

J. M. Plowman, *The Reinforced Concrete Review* (London), V. 4, No. 3, Sept. 1956, pp. 177-220

Includes a review of research and construction practices involving vibration of concrete, its effect on bond to reinforcement, mix design for vibration, measurement for vibration within concrete, and concludes that this field requires much more research to take better advantage of vibration for improving concrete. Twenty pages of discussion bring out several particular problems in vibrating concrete.

Structural Research

Properties of anchor bolts (in Finnish)

V. A. A. Ahtee, *Rakennusinsinööri* (Helsinki), V. 12, No. 3-4, 1956, pp. 32-36

Reviewed by B. Kelopuu

Author describes tests carried out with anchoring bolts in rock with special attention to following items: influence of grouting

length on strength of anchorage; possibilities of using deformed bars as anchoring bolts without special anchoring devices; influence of diameter of grouting hole; and influence of strength of grouting mortar.

Failure was caused by fracture in rock, by exceeding bond strength between grout and rock or steel and grout.

Grouting length depends mostly on the bond between steel and grout. Only in case of water breaks in rock—when cement in grout can be washed away—bond between grout and rock is of importance.

Grouting length of 25 diameters in bolts with wedged heads has been found sufficient. Deformed bars can be used without wedged heads with grouting length of 20 diameters. Quality of mortar seems to have influence. A mix of 1:1 (sand $< \phi$ 1 mm) and $W/C = 0.5$ is recommended. Diameter of grouting hole seems to be of little influence.

Practical method of analyzing structures using large models

K. F. Tse, *Civil Engineering and Public Works Review*, (London), V. 51, No. 606, Dec. 1956, pp. 1341-1344

Describes simple method and apparatus designed to induce and control large distortions for ease in measurement on models used for structural analysis. Paper is first part of a continuing series.

Tests on circular-pipe-culvert inlets

R. E. Schiller, Jr., *Bulletin* No. 126, Highway Research Board, Sept. 1956, pp. 11-23

Model tests were carried out with six different types of inlets. Test results are indicated and the following conclusions are drawn: (1) efficiency is reduced if upstream end is mitered as compared to square-end inlet formed by placing groove end of tongue-and-groove pipe upstream; (2) most of pipes tested showed that culverts would flow full for inlet conditions only when head is 1½ times the depth of the culvert; (3) the sharp-edged projecting inlet and the mitered sharp-edged inlet are not as efficient as the square-edged flush inlet. Results of flow tests for different types of inlet are presented as well as design examples based upon the conclusions drawn. Includes discussion which points out limiting conditions within which conclusions may be applied.

General

Surge-wave velocity—concrete pressure pipe

H. F. Kennison, *Transactions*, American Society of Mechanical Engineers, V. 78, No. 6, Aug. 1956, pp. 1323-1328 (including discussion)
Reviewed by Aron L. Mirsky

Surge-wave velocity in the case of reinforced or prestressed concrete pressure pipe is a function of modulus of elasticity of pipe, which is not a constant but a function of the maximum water-hammer pressure. Paper outlines method of calculating surge-wave velocities for such pipes, and reports results of some tests which substantiate method.

Concrete and masonry

E. Stieri, Barnes and Noble, Inc., New York, 1956, 277 pp. $1.75

A reference book on concrete for the "do-it-yourself" home builder. Sets forth principles of concrete making in a simple way with directions for solution of common problems involved. Includes description of simple form making for usual home projects, directions for stucco and concrete masonry work, fairly complete specifications, and plans for a variety of home projects.

Demonstration of possible flow conditions in a culvert

M. R. Carstens and A. R. Holt, *Bulletin* No. 126, Highway Research Board, Sept. 1956, pp. 1-10

Possible flow profiles within a highway culvert were reproduced in a two-dimensional model, photographed, classified, and analyzed in detail. Presents a qualitative study on the effects of entrance streamlining, barrel length and roughness, and outlet submergence.

Vibrating tables

W. K. Sneddon, *Constructional Review* (Sydney), V. 29, No. 6, June 1956, pp. 18-22

Application of the theory of dynamics to the design of vibrating tables for concrete products. Includes practical comments and conclusions for optimum operation in minimum time to produce best quality product.

Hydraulic self-aligning seating

A. C. Carter, *The Engineer* (London), V. 202, No. 5247, Aug. 17, 1956, pp. 232-234
Reviewed by Aron L. Mirsky

Covers design and development of a hydraulic testing head for testing of concrete cubes. Object was to produce equal *stress*

(rather than equal *strain*, such as is produced by fixed platens) in the specimens. Grinding of cube faces is obviated and localized stresses are avoided using the hydraulic self-aligning head.

Timber design and construction handbook

Timber Engineering Co., F. W. Dodge Corp., New York, 1956, 622 pp., $12.75

A comprehensive timber design reference as well as a practical field handbook. Combines many of the features of separate earlier publications by the Timber Engineering Co. in one volume. It is divided into three main sections: (1) fundamental characteristics of wood; (2) design, detailing, fabrication, and erection recommendations; (3) design and engineering specifications and tabular data in simplified form. A special short section deals with problems of form design for concrete shells. This new book should be of interest to any structural engineer or to any construction engineer dealing with concrete construction as well as with wood construction.

Application of atomic research to building (Application des Recherches Atomiques au Batiment)

Col. Feckler, *Cahiers*, Centre Scientifique et Technique du Batiment (Paris), *Bulletin* 214 (no date), pp. 27-29

Reviewed by Edmund A. Pratt

Describes activities in laboratory of the center in the application of x-rays and gamma radiation (radiocobalt) for evaluation of defects in concrete (setting, consolidation, homogeneity, location and size of reinforcement) and for systematic research on determination of water content and relative densities obtained by vibration. Some information is given on energy of radiation and time of exposure for various thicknesses of concrete.

Architectural graphic standards

Charles G. Ramsey and Harold R. Sleeper, John Wiley & Sons, Inc., New York, 5th Edition, 1956, 758 pp., $18.50

A new edition of a familiar work, consisting entirely of illustrations, marked by thorough notations. Some 236 plates have been revised for the fifth edition and 286 new plates added; material has been rearranged in 23 major groups.

New material includes: design of plank and beam framing; curtain walls; pneumatic tubes; elevators and escalators; design of special fireplaces; comparative costs of roof covering; furniture and equipment; and tile and its correct usage.

Subjects of the revised sections are: swimming pools; metal lath and plastering; doors and windows; architectural terra cotta; metal facing materials; bar joists; lally columns; glued and laminated members; reinforced concrete; roofing materials; interior finishes; acoustical correction; insulation values of exterior walls; plumbing; electrical equipment; kitchen fixtures; and sports and games.

Fire protection equipment list

Underwriters' Laboratories, Inc., Chicago, Jan. 1957, 283 pp.

Includes official retardant classifications (fire ratings) for assemblies of various building materials including structural steel, reinforced concrete, plasters, sheet steel, etc., as columns, walls, slabs, for floor, roof, and ceiling construction and beams. Includes list of concrete masonry manufacturers certified by Underwriters Laboratories, Inc.

The impact of engineering on society

Maurice Bowra, *Proceedings*, Institution of Civil Engineers (London), Part I, V. 5, No. 5, Sept. 1956, pp. 633-644

Reviewed by Aron L. Mimsky

In these days when almost every engineering journal carries articles exhorting engineers to be professional men, a paper like this, literate, well-reasoned, examining the role of the engineer in the world from the earliest times to the present and the various "objections" to him and to his activities and effects on social life, is most welcome, even though—or perhaps especially because—it is by a non-engineer who confesses that he cannot even drive a car. (He *can* spell, which the authors of many of the afore-mentioned exhortations apparently cannot.)

Structures

Pier Luigi Nervi, F. W. Dodge Corp., New York, 1956, 136 pp., $6.95

An exciting challenge to imagination of both architect and structural engineer. Includes many construction views and completed illustrations of Nervi's works. A brief explanation is given of construction materials and methods, also methods of design and analysis used by Nervi to accomplish his

unique results economically. Admitting the difficulty of transmitting intuitive design attitudes to others, Nervi suggests that engineering and architectural schools make much more use of models and explains his use of model analysis to enable utilization of complex structural concrete.

Lectures on science of construction. Part I—Elementary theories of elasticity, resistance of materials (Lezioni di scienza delle construzioni. Parte prima—Elementi di teoria dell' elastica, resistenza dei materriali)

L. F. Donato, Colombo Cursi, Pisa, Italy, 3rd Edition, 1955, 282 pp., 3200L
Reveiwed by J. J. Polivka

Only minor revisions have been made in this third edition of interesting textbook being used by Professor Donato in Italy. Sixteen chapters deal with: mechanics of deformable bodies; analysis of deformations; equilibrium of solid cements; potential elastic energy; theories of elastic equilibrium (Clapeyron, Betti, Maxwell, Menabrea, Castigliano, second principle of deformation, De Saint-Venant); isotropic solids; two-dimensional elastic systems; general aspects of De Saint-Venant's problem; action of simple normal forces; simple bending; simple torsion; combined bending; eccentric normal forces; other combined forces; behavior of materials under loading; criteria of resistance and safety.

Solution of difficult foundation problems of bridges in saline soils (in Rumanian)

M. Radulescu, *Industria Constructiilor si a Materialelor de Constructii* (Bucharest), V. 7, No. 3, March 1956, pp. 141-145
Reviewed by J. J. Polivka

Foundation difficulties in saline soils under water-saturated strata are described and their solution presented in connection with construction of a railroad bridge. Special type of concrete caisson was used.

Elementary differential equations

T. W. Martin and E. Reissner, Addison-Wesley Publishing Co., Reading, Mass., 1956, 260 pp.
Reviewed by Gunhard A. Oravas

It becomes more and more important for progressive engineers, who are interested in taking active part in the professional development, to follow new theoretical research in engineering literature. The major obstacle that limits the circle of readers of theoretical papers seems to be that most engineers find themselves helpless when confronted with a differential equation.

The distinction of this book is that the developments of solutions for differential equations have been accomplished by extreme simplicity and with an attitude toward the subject matter that ideally suits the needs of practical engineers.

The first chapter deals with formulation and origin of differential equations in connection with geometry and mechanics. Chapter 2 treats differential equations of first order, method of separation of variables, integration of factors, variation of parameters, power series methods, nonlinear equations, method of undetermined coefficients, convergence of series solutions.

Chapter 3 deals with second order differential equations following the same pattern of development as in Chapter 2. In addition power series solutions are developed for regular singular points. Bessel equation receives special treatment. Chapter 4 treats higher order differential equations and also Laplace transformation method of solution.

Chapter 5 handles systems of differential equations. Chapter 6 considers approximate solutions of first order differential equations; Picard's iteration and Taylor series methods are handled at adequate length.

Chapter 7 shows the application of finite difference equations in connection with the solution of differential equations and linear difference equations.

Chapter 8 gives a short treatment of partial differential equations that concludes this concise and very practical treatise.

Throughout text, equations are handled in a spirit of engineering application. The great importance of the book lies not only in its simplest possible explanations for the solutions, but also in the extremely elementary manner in which some insight is given to the uninitiated reader about their range of applicability. Power series solutions of differential equations are masterpieces of simple deductive reasoning from the practical engineer's point of view. The junior author, being one of the foremost authorities on the theory of thin shells and cognizant of the

problems of struggling engineer, has evidently exercised an important influence on the excellent choice and form of the subject matter.

This text has included a selection of topics, all treated to the extent of practical significance, which in reviewer's opinion is adequate to provide a well balanced know-how in the application of differential equations. This text is eminently suited for self-instruction. The authors have distinguished their book with a quality of shrewd and yet elementary mathematical presentation that should, at least from the vantage point of a practical engineer, make this volume stand out among other available textbooks on the theory of differential equations.

Modern masonry (natural stone and clay products)

Publication No. 466, Building Research Institute, Washington, 1956, 161 pp., $4.50

The edited papers and discussions of a research correlation conference on natural stone and clay products conducted by the Building Research Institute in Washington, D. C., Sept. 19 and 20, 1956. Of particular interest to structural engineers is the paper included on reinforced brick masonry, pp. 31-46, which reviews practice, theory, and applications of reinforced, grouted brick masonry.

A second article, "Maintenance of Industrial Buildings," pp. 103-114, includes many examples of typical crack conditions in masonry walls, principally due to common faults in design, and describes maintenance and repair procedures required to correct the objectionable conditions.

Next Month . . . . .

The June Current Reviews section will carry the annually published list of

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including the publisher's address (in some cases the name also). This is for the convenience of readers who may desire copies of the books and magazines reviewed, since they are NOT available through ACI.